

Emilio C. Cronin

CONNECTICUT RIVER FLOOD CONTROL PROJECT

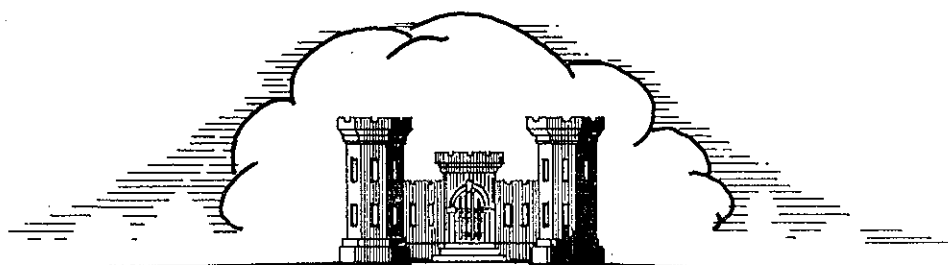
ENGINEERING DIVISION WORKING COPY
RETURN TO FILE

CLAREMONT DAM

SUGAR RIVER, NEW HAMPSHIRE

ANALYSIS OF DESIGN

APPENDIX A



WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY
U. S. ENGINEER OFFICE PROVIDENCE, R. I.

JANUARY 1945

CONNECTICUT RIVER FLOOD CONTROL
ANALYSIS OF DESIGN

CLAREMONT DAM
NEW HAMPSHIRE

APPENDIX A

FOUNDATION TREATMENT WITH DRAIN WELLS AND RELIEF WELLS

JANUARY 1945

CORPS OF ENGINEERS, U. S. ARMY

U. S. ENGINEER OFFICE

PROVIDENCE, RHODE ISLAND

CLAREMONT DAM
ANALYSIS OF DESIGN

APPENDIX A

FOUNDATION TREATMENT WITH DRAIN WELLS AND RELIEF WELLS

	<u>Contents</u>	<u>PAGE</u>
A.	<u>INTRODUCTION AND SCOPE</u>	1
B.	<u>FOUNDATION CONDITIONS</u>	3
	1. Foundation soils	3
	2. Undisturbed sampling	4
	3. Properties of soft, varved silt	5
C.	<u>STABILITY ANALYSIS - CONSOLIDATION BY VERTICAL FLOW ONLY</u> . . .	13
	1. General	13
	2. Assumptions	13
	3. Analysis method	14
	4. Results - Downstream slope	17
	5. Reliability of results	18
D.	<u>FUNCTION OF DRAIN WELLS</u>	23
	1. Proposed installation	23
	2. Advantages of drain wells	23
	3. Prior use	24
	4. Theory	25
E.	<u>STABILITY ANALYSIS WITH IDEAL DRAIN WELLS</u>	26
	1. Case considered	26
	2. Analysis method	26
	3. Analysis results	32
F.	<u>EFFECT OF WELL RESISTANCE</u>	34
	1. Well resistance	34
	2. Well backfill	34
	3. Mathematical solution	35
G.	<u>EFFECT OF SMEAR</u>	37
	1. Description of smear	37
	2. Mathematical solutions	37
	3. Importance of smear reduction	40

	<u>Contents (cont'd)</u>	<u>PAGE</u>
H.	<u>WELL STIFFNESS</u>	42
	1. Deformation of wells	42
	2. Past experience	43
	3. Probable well action	44
I.	<u>SELECTION OF WELL SPACING AND DIAMETERS</u>	45
	1. General	45
	2. Favorable features	45
	3. Unfavorable features	46
	4. General considerations	46
J.	<u>DRAIN WELL CONSTRUCTION METHODS</u>	48
K.	<u>OBSERVATION DEVICES</u>	51
	1. Piezometers	51
	2. Settlement gages	51
	3. Lateral movement	52
L.	<u>RELIEF WELLS</u>	53
	1. Deeply buried pervious deposit	53
	2. Uplift pressure - no wells	53
	3. Effect of high uplift pressures	55
	4. Proposed treatment	55
	5. Relief well details	56
	6. Relief well discharge	57
	<u>INDEX OF TABLES AND PLATES</u>	59

CLAREMONT DAM
ANALYSIS OF DESIGN
APPENDIX A

FOUNDATION TREATMENT WITH DRAIN WELLS AND RELIEF WELLS

A. INTRODUCTION AND SCOPE.

1. The site of the proposed Claremont Dam is situated on the Sugar River about 1 mile above the City of Claremont, N. H. This will be an earth dam having a maximum height of 128 feet and about 2600 feet long and will be a retarding reservoir unit of the Connecticut River Basin flood control project. General Plan and typical sections are shown on Plates Nos. A1, A2, and A3. Vicinity map is included on Plate No. A4.

2. The foundation has been explored by numerous borings as shown on Plate No. A4. The abutments consist mainly of compact glacial till or rock. The flood plain, which is about 1300 ft. wide, contains a deep deposit of soft, varved silt which creates a major foundation problem of stability. Presence of a deep pervious layer at valley bottom requires consideration from the standpoint of flotation.

3. Material presented in this Appendix covers analysis of these foundation problems of stability and flotation and treatment by drain wells and relief wells, respectively. Available data are included on properties of the soft, varved silt deposit and typical stability analyses, with and without foundation treatment.

a. The theory of consolidation with normal vertical drainage aided by radial drainage to drain wells has been presented in a Providence District publication (1) and is herein expanded to cover

(1) R. A. Barron (1944) "The Influence of Drain Wells on the Consolidation of Fine-Grained Soils"; U.S. Engineer Office, Providence, R.I.; July 1944.

effect of flow resistance within the wells and effect of smear reducing permeability at periphery of the well. Stability analysis is carried out as an example of application of this theory in design of a system of drain wells.

B. FOUNDATION CONDITIONS.

1. Foundation Soils.

a. The flood plain foundation at the dam site consists in general of 15 to 20 feet of typical flood plain alluvial deposits of silts, sands, and gravels deposited in layers and lenses and being generally pervious to random pervious. Beneath this alluvium is a deep impervious deposit of soft varved silt, 40-100 feet in thickness, consisting of silt interstratified with fine sand and some lean clay. Underlying this soft silt deposit is an impervious layer of very compact, silty, glacial till which alternates with and frequently overlies a layer of compact varved silt. At the very bottom of this soil deposit is a bed of pervious sands which lies just above the bed rock in the preglacial rock valley. Foundation conditions in the flood plain area are shown on Plates Nos. A5, A6, and A7, and borings are located in plan on Plate No. A4.

b. The right abutment consists mainly of a thick, impervious deposit of very compact silty glacial till overlying a thinly bedded, siliceous sericite schist. The left abutment is very similar, except that the deposit of till is much thinner as shown on Plate No. A5. Both abutments have local deposits of compact varved silts very similar to that found deep in the flood plain deposit. Both upstream and downstream are remnants of a river terrace composed of sands and gravels.

c. Based upon a study of the soil profiles and samples, it appears that this area contained a glacial lake in which the varved silts were deposited. Fluctuation of the ice front resulted in the

removal of some of this silt and the compaction of the remainder with deposition of till above. The soft, varved silt was probably deposited in the last glacial lake and has probably never been subjected to any greater load than that represented by the former river terraces, of which remnants remain on the abutments.

2. Undisturbed Sampling.

a. Because of the looseness and essentially cohesionless character of the soft, varved silt, some difficulty was experienced in obtaining, transporting, and testing the undisturbed samples. Samples were obtained in 2" inner diameter Shelby tubes without too much difficulty, although the recovery was often not quite 100 percent and an occasional sample was lost. Larger undisturbed samples for consolidation and direct shear testing were obtained in 4-3/4 inch inner diameter brass tubes. With these larger sampling tubes much more difficulty was experienced and due to cohesionless nature of the silt, bulk of these samples was lost before tubes were removed from the borings. Samples successfully recovered in the 4-3/4 inch spoon therefore tended to be those with a greater concentration of varves of cohesive lean clay. For final sampling a freezing unit sampler was resorted to and by freezing a plug at sample end, 2.8 inch diameter samples were successfully obtained for triaxial tests. Sampling procedure is described in a previously issued Providence District publication (2).

b. The samples were all transported from the site to the District Soils Laboratory at Providence, R. I. by truck. Samples were placed vertically in a special carrying rack that rested upon soft

(2) F.E. Fahlquist, 1941, "Undisturbed Sampling of Sediments"; U. S. Engineer Office, Providence, R. I.; November 1941.

mattresses to reduce vibrations from truck floor. In all cases the truck was operated with special care to reduce vibrations and shocks to a minimum. In spite of these precautions some consolidation of the samples occurred as indicated by a small amount of free water at top of samples. One or two samples were carried in a horizontal position and upon arrival a water filled void was found running the entire length of tube, making these samples useless for testing.

3. Properties of Soft, Varved Silt.

a. Classification. - A large number of 2 inch Shelby tube samples were extruded from the sample tubes in short sections 3 to 5 inches long. These short sections were split in two; one half being used for water contents and void ratio while the other half was partially air-dried to bring out soil varves by the contrast in color of the different soils when partially dried. These samples were then classified visually in accordance with the standard Providence District Classification System as given on Plate No. A8 and Table No. A1. In general, the soil consists of medium to coarse silts interstratified with layers of fine silty sand, coarse silts and some lean clays and has been termed varved silt as it is basically cohesionless. Photographs of typical varved silts are shown on Plates A9 and A10.

(1) Because of the stratified nature of the soft silt and because of the small thickness of each varve, a grain size determination would be a composite of more than one soil. A few of these were run, however, and also a few on some of the thicker varves which could be identified after partial drying. The range for these materials is shown on Plate No. A32.

b. Water Content and Void Ratio. - Water content, void ratio, and dry density were determined from remaining half of split Shelby tube samples. Results from these 3 to 5 inch sections were averaged for each undisturbed sample and plotted to obtain general variation of properties with depths. Void ratios and dry densities are based upon the average specific gravity of each section and full saturation. Typical examples are shown on Plates Nos. A11 and A12.

(1) In the natural state the average void ratio of the soft varved silt ranges from 0.80 to 1.20 with an overall average of about 0.95. This results in a range of dry density of 77 to 92 lbs/cu.ft. and an overall average of about 87 lbs/cu.ft. The water content ranged from 30 percent to 40 percent of the dry weight with an overall average of about 35 percent.

(2) Occasionally it was feasible to distinguish and obtain water contents on individual varves, which for the silt alone ranged from 20 percent to about 25 percent. The data on lean clay give an average of about 60 percent and the fine sand is estimated at from 10 percent to 15 percent water content.

c. Consolidation.

(1) Consolidation test specimens were obtained mainly from undisturbed samples taken in 4-3/4 inch brass tubes from bore holes BH-46, 47, 70, and 100 while a few additional specimens were obtained from 3-inch Shelby tubes from BH-100. Tests were run in fixed ring consolidometers as the soil was too soft to support a floating ring. Specimens of 4-1/4 inch diameter were loaded in increments to 8 tons/sq. ft. To better investigate probability of preconsolidation a few

specimens were loaded to 16 tons/sq.ft., using 3 inch diameter specimens to obtain this higher unit load within capacity of available apparatus. Test results indicated that the varved silt has widely varying properties, depending upon the major type of soil in the varves of the specimen tested.

(2) The coefficient of consolidation, C_v , ranged from 10×10^{-4} to $5000+ \times 10^{-4}$ cm²/sec. with an average value of 35×10^{-4} cm²/sec. for a load of 5-1/2 tons/sq.ft. which approximately equals the sum of average dam and overburden loads. C_v is defined as:

$$C_v = \frac{T \left(\frac{H}{1+e} \right)^2}{t}$$

Where; "T" is time factor, "e" is initial average void ratio, "H" is the thickness in centimeters of sample for case of single drainage, and "t" is time in seconds. Typical consolidation characteristic curves are shown on Plates Nos. A13 to A18 inclusive.

(3) As shown on Plate No. A1, remnants of a former river terrace exist at about Elevation 565 which is about 30 feet above the present flood plain and it is probable this river terrace extended across most of valley. Using an average moist weight of 130 lbs/cu.ft. this is the equivalent of a load of 1.95 tons/sq.ft.; however, the water table probably was such that all of this 30 feet of extra soil was not moist but rather partly submerged which would reduce the net load of the soil that has been removed. Pre-consolidation results obtained from consolidation tests on undisturbed samples are summarized on Plate No. A19 and show a wide scatter with an average of 1.45 tons/sq.ft. These results are quite approximate because low capacity of consolidometer

loading machines did not permit full development of virgin consolidation curve and probably because of some disturbance during sampling. However, the average test results and geologic evidence indicate that the soft silt was pre-consolidated by past load of the river terrace, probably of the order of 1-1/2 tons/sq.ft.

d. Permeability.

(1) Permeability values of the soft varved silt were obtained indirectly by computation from consolidation test results. Typical examples are given on Plates Nos. A14, A15, and A18. For an average dam and overburden load of 5-1/2 tons/sq.ft., the values range from 1.4×10^{-8} to over 500×10^{-8} cm/sec. All of these results are for flow normal to the plane of stratification - i.e. k_v vertically.

(2) Permeability in the horizontal direction parallel to the plane of stratification, k_H , was not tested. However, when later stability studies indicated that drain wells were necessary to stabilize the foundation, a rough value was obtained as shown on Plate No. A20. This value was obtained from consolidation test results for permeability in a vertical direction, k_v , by assuming that the results obtained were representative of the distribution of permeability values and that for each test specimen the ratio $k_H/k_v = 1$. This is the equivalent of placing the 17 consolidation test specimens in a pile, one above the other, and assuming each specimen homogeneous in itself and that the result approximates a cross section of the varved silt deposit. The assumption is crude, and the consideration of $k_H = k_v$ for each specimen is quite conservative.

(3) The overall k_v and k_H were then found by use of the formulas:

$$k_v = \frac{n}{\sum \frac{1}{k}} = 5.74 \times 10^{-8} \text{ cm/sec.}$$

$$k_H = \frac{\sum k}{n} = 79.6 \times 10^{-8} \text{ cm/sec.}$$

Where all the test specimens are of equal thickness and n is the number of specimens, the ratio k_H/k_v is then equal to 14.0%. Because of the assumption used this ratio is believed to be conservative and the true value of the $\frac{k_H}{k_v}$ ratio may be much higher.

(4) More recently permeability tests have been run on undisturbed samples of rather similar varved silts, from two other sites. The k_H/k_v ratio obtained ranged from 5 to 80 with an extreme value of about 3000. This latter value is somewhat questionable, but the general results give an idea of the probable magnitude of the k_H/k_v ratio in such a strongly stratified soil as this varved silt.

e. Direct Shear Tests. - A number of direct shear tests were run on specimens of undisturbed, soft, varved silt obtained in 4-3/4 inch brass tubes. Specimens were placed in shear boxes between porous stones and fully consolidated under a predetermined normal load. When consolidation under normal load was complete, the shear tests were started. Horizontal strains were applied in small increments and specimens allowed to reach void ratio adjustment under each increment before the next was added so that specimens were practically always fully consolidated under total shearing and normal loads. Summary of results are given on Plate No. A21. From these "slow" tests values of friction angles vary from 27-1/2° to 36° with cohesion of 0.0 to 0.12 tons per sq. ft.

f. Triaxial Tests. - A few "slow" triaxial tests were run on undisturbed samples of the soft, varved silt where the specimens were nearly always fully consolidated under external loads. Generally, the material was so soft and loose that considerable difficulties were experienced in preparing and setting up test specimens; many specimens would deform by bulging even before being set up and were therefore discarded. Samples that were set up either had sufficient clay to furnish cohesion or were partially air-dried to obtain apparent cohesion from capillarity to permit preparation of specimens for testing.

(1) Only six triaxial tests were obtained from results that were considered at all reliable. The specimens were 1.4 inches in diameter and about 4 inches high, tested by the constant lateral pressure method in a constant load device. Specimens were all consolidated initially under uniform lateral pressure; then axial loads were added in increments, always allowing complete consolidation under each increment before the next was added. Time for each such slow test averaged about 3 weeks. Maximum strengths were obtained between 10 percent and 16 percent strain with friction angles ranging between 33° to 42° with very little or no cohesion - see Plate No. A22 for Mohr's circles and Table No. A2 for data. Because of difficulty in handling soft silt, those specimens tested are apt to have been either the stronger of the silt or strengthened by partial drying before being able to trim and handle for testing.

(2) The above tests were performed between January and June of 1941 and the samples at that time were between 1/2 and 1 year old. Because of war conditions further testing was suspended

until March 1944. The samples had been stored in the original steel sampling tubes in the laboratory humid room during this 3 year interval and were found to be frequently rusted on the inside to form a cementing layer between tube and silt. Partly for training personnel the less damaged remaining samples were subjected to quick-consolidated tests. These latter tests were all consolidated under initial uniform pressure and then tested rapidly to failure without allowing further drainage. Results gave a range of friction angle between 10° and 25° and cohesion between 0.3 to 0.4 tons per sq. ft. Because of age and physical conditions of samples at time of tests these latter results are not considered to be reliable.

(3) Sufficient triaxial tests were not conducted to estimate critical void ratio and density. Considering general loose state of this silt, it is probable that material in the silt and fine sand varves is looser than the critical density.

(4) Because of difficulties experienced in preparing and setting up test specimens the triaxial studies were continued to develop a reliable method of handling such a soft varved silt. A scheme was developed whereby the soft silt would be ejected vertically out of the sampling tube into a test rubber membrane 2.8 inches in diameter which is the same as inner diameter of sampling tube and thus eliminates need for trimming. Then by means of special forms the sample would be transferred to testing machine while continuously supported, and set up ready for testing with a minimum of disturbance. Experimental equipment was built and tried on varved silt samples obtained from Keene, N. H., where the deposit is very close to the surface. The scheme with

some modifications was reasonably successful and practical. However, further triaxial testing of Claremont silts was not continued because expensive new samples would be required and because stability studies indicated that with drain wells the proposed design would be conservative for soils having a friction angle of 25° to 30° .

C. STABILITY ANALYSIS - CONSOLIDATION BY VERTICAL FLOW ONLY.

1. General. - Because of the very limited knowledge of shearing strength of the soft varved silts, stability analyses were made to determine the required shearing strength of the silt for a safety factor of 1.0 for dam and foundation against shear failure by a slide. Because the fully consolidated shear tests, both direct shear and triaxial, indicated very little cohesion, which was also apparent from behavior of samples, no cohesion was considered in these studies.

2. Assumptions.

a. Average soil properties and unit weights of dam embankment soils and foundation soils as determined by testing and ground water level, are shown on Plate No. A24.

b. Although the rolled fill method of embankment placement may give higher horizontal earth pressures initially within the embankment, it was assumed that there will be sufficient yield of the weaker foundation for a stability safety factor of 1.0 to reduce such to active earth pressure within the dam embankment. This results in assumption of a state of active earth pressure on sliding surfaces within dam itself.

c. It was assumed in initial computations that full passive earth pressure will be developed at the toe of any potential slide in the overburden over the silt. In some cases the resulting passive pressure furnished a fair percentage of the resisting forces. This assumption was later revised and computations corrected to allow for only 50 percent of full passive earth pressure at slide toe.

d. The stability studies were based upon an initial rate of embankment loading to elevations shown on Plate No. A5, at rates uniform between each elevation.

e. It was assumed that all excess water expelled from the soft silt in the foundation by the addition of the dam load, escaped by flow in a vertical direction only. That is, the bottom of the soft silt was assumed to be impervious with case of single drainage and horizontal permeability was neglected. The soft silt is underlain by quite impervious glacial till or by very compact varved silt. The latter silt was considered more impervious than the soft silt deposit because of increased compactness.

f. The rate of consolidation was controlled by thickness of the soft silt deposit and by the average value of coefficient of consolidation, $C_v = 35 \times 10^{-4} \text{ cm}^2/\text{sec.}$ as discussed above in Paragraph B3c (Page 6). Allowance for varying silt thickness was made by computing consolidation rate at several different points.

3. Analysis Method.

a. It was assumed that potential failure surfaces would be planes in granular foundation overburden above soft varved silt and in dam embankment and circular arcs in the soft varved silt. Consideration of plane sliding surface in embankment is consistent with the state of active earth pressure assumed therein. The extent of any sliding segment was considered to be sufficiently long parallel to the dam centerline to permit any three dimensional aspects to be neglected.

b. Total stresses at any point on a potential failure surface was computed as the vertical dimensions of the different over-

lying materials multiplied by their unit weights. This neglects any spreading or arching of the dam load. For the first season's construction this is probably close to actual stress condition. For second and third season result is more approximate, although because of flat slopes used the approximations are probably fairly close. Normal and tangential components of vertical forces were determined graphically, as shown on Plate No. A24.

c. The effect of consolidation was determined for a number of points in the foundation (assumptions "e" and "f" in Paragraph C2 above) and contours of equal hydrostatic excess pressure were constructed for the end of each construction season as shown on Plate No.

A25. The rate of loading was considered by dividing the load into infinitesimal increments and the percent consolidation for each increment was determined and integrated graphically as shown on Plate No. A26.

The method of obtaining these contours of excess pressure with allowance for effect of gradual load application is illustrated in Section E, Paragraph 2, page 26.

d. From the hydrostatic excess pressure contours the excess hydrostatic pressures were determined along any potential failure surface. These pressures were then added to ground water or hydrostatic pressure to give total neutral stress in the pore water. This total neutral stress was then deducted from the total normal pressures to obtain the effective or intergranular stress as shown graphically on Plate No. A24.

e. The method used in the stability analysis was the method of slices having an infinitesimal thickness as given by May (3).

(3) D.R. May - (1938) "Application of the Planimeter to the Swedish Method of Analyzing the Stability of Earth Slopes"; Transactions, Second Congress on Large Dams, Vol. IV, p. 540; Washington, D. C.; 1938.

(1) Total vertical weight of slices located at various points along the assumed failure arc was found using moist unit weights above water table and saturated unit weights below - this force being noted as " σ_t ".

(2) The uplift effect of water pressure due to natural ground water (hydrostatic pressure) plus that due to any hydrostatic excess pressures as determined from excess pressure contours were added together to obtain total neutral stress " σ_n ". This force was then subtracted from the total vertical force " σ_t " to obtain the effective vertical intergranular force " σ_{eff} ".

(3) To determine the overturning forces the force " σ_t " was resolved into a component or force vector "T" tangent to the failure arc. To determine the shearing resisting force the force " σ_{eff} " was resolved into a force vector " N_{eff} " normal to the failure arc. The values of " N_{eff} " and "T" were then plotted on vertical lines projected up from points on the failure arc to obtain diagrams shown on Plate No. A24.

(4) The overturning moment about the failure arc center was then determined by obtaining the area of the "T" diagram with a planimeter and converting it into a moment by multiplying the area by scale factors of the diagram and by the failure arc radius. The remaining portion of the overturning moment due to active earth pressure acting horizontally within the dam was then found using the formula for active earth pressure. The moments of these forces were then added to that due to sum of "T" forces to obtain total overturning moment.

(5) The resisting moment was found by summing up the "N_{eff}" forces in a manner similar to that used for "T" forces. This total force was multiplied by tan ϕ of silt and by radius of failure arc to obtain resisting moment of shearing forces along failure arc. To this moment was added that due to horizontal passive earth pressure at toe of failure arc where it emerged from the silt into the sand and gravel overburden.

(6) The factor of safety against a slide can then be expressed by the following formula, assuming the silt has no cohesion.

$$F.S. = \frac{\int "N_{eff}" \cdot R \cdot \tan \phi + \text{Moment passive pressure at toe}}{\int "T" \cdot R + \text{Moment active earth pressure in dam}}$$

Where R is the radius of assumed failure arc. Because of limited knowledge of shearing strength of soft varved silt the required friction angle ϕ in the silt was carried as the unknown and determined for F.S. = 1.00.

4. Results - Downstream Slope.

a. Thickness of the soft varved silt deposit is a maximum under the downstream slope and therefore, during the construction period, this area will be more critical because of its slower rate of consolidation. For a safety factor of 1.00 and neglecting any minor amount of cohesion that may be present, the stability studies indicate the following required shearing angle in the soft varved silt to the nearest half degree:

End of 1st construction season - 16-1/2°

End of 2nd construction season - 16°

End of 3rd construction season - 18°

b. The stability analysis method used requires numerous trials to locate the most critical failure surface. Several failure

surfaces were analyzed (approximately 3 or 4 for each case studied); but due to limited personnel the number of failure arcs analyzed was not necessarily sufficient to locate the most critical one. Because the actual maximum required shearing angle may be slightly larger than those listed above, 2° additional has been estimated to allow for this. Further, full passive earth pressure was used in computations at toe of failure surface where it passes from the soft varved silt to the overlying sands and gravels. This may not be on the safe side and a re-computation showed the required shearing angle for the silt would be raised about 1° by using only half of full passive pressure. Therefore for a safety factor of 1.0 the required computed shearing angles in the soft silt have been arbitrarily raised 3° to the following values:

End of 1st construction season - $16\frac{1}{2}^\circ + 3^\circ = 19\frac{1}{2}^\circ$

End of 2nd construction season - $16^\circ + 3^\circ = 19^\circ$

End of 3rd construction season - $18^\circ + 3^\circ = 21^\circ$

5. Reliability of Results. - The above analyses have been predicated on the assumption of drainage during consolidation only in a vertical direction which is reasonable only for homogeneous compressible deposits. In materials as strongly varved as the Claremont silt, the horizontal permeability is much greater in the direction of the varves than the vertical permeability. In such case a very considerable amount of drainage occurs in a horizontal direction with accompanying transmission of excess pressure laterally in the more pervious varves.

a. Lateral transmission of hydrostatic excess pressure or unbalanced pressure in the silt may be very dangerous from the standpoint of stability in the sense that it will tend to lift the soil under the toe of the dam and reduce shear strength there.

(1) The process of consolidation is well illustrated in the following equations for stress in the soil:

$$\begin{array}{rcccl} \sigma_{\text{total}} & = & \sigma_{\text{initial}} & + & \Delta \sigma_{\text{added}} & + & u' & \text{hydrostatic pressure} \\ & & \downarrow & & \downarrow & & \downarrow & \\ & = & \sigma_{\text{effective}} & + & u & + & u' & \text{hydrostatic pressure} \\ & & \text{or} & & \text{pore water} & & & \\ & & \text{intergrain} & & \text{excess} & & & \\ & & & & \text{pressure} & & & \end{array}$$

At the instant of loading effective stress ($\bar{\sigma}_{\text{eff}}$) equals only the initial stress (σ_i) and the added load ($\Delta \sigma$) goes into water pressure as (u) the hydrostatic excess pressure - the hydrostatic pressure (u') remaining unchanged. The process of consolidation involves squeezing out of the excess pore water, transferring the stress, u , to $\bar{\sigma}_{\text{eff.}}$, until at the end of consolidation:

$$u = 0$$

and

$$\bar{\sigma}_{\text{eff.}} = \sigma_i + \Delta \sigma$$

Only the effective stress ($\bar{\sigma}_{\text{eff.}}$) or intergrain stress creates shearing strength in the soil by internal friction.

(2) Beneath the centerline of the dam the added stress, $\Delta \sigma_c$, is high and much greater than that at the toe, $\Delta \sigma_e$, with corresponding difference in the hydrostatic excess pressures.

Assume for comparative purposes

$$\Delta \sigma_c = \Delta \sigma_e + \beta$$

Whence, initially $u_c = u_e + \beta$

Then at toe, at instant load is applied:

$$(\sigma_t)_e = (\sigma_i)_e + \Delta \sigma_e + u'$$

But if the excess pressure at center is transmitted laterally in the more pervious varves of fine sand, conditions at toe could then become:

$$(\sigma_t)_e = (\bar{\sigma}_{eff})_e + \Delta\sigma_e + \beta + u'$$

and

$$(\bar{\sigma}_{eff})_e + \Delta\sigma_e + \beta + u' = (\sigma_i)_e + \Delta\sigma_e + u'$$

$$(\bar{\sigma}_{eff})_e = (\sigma_i)_e - \beta$$

Whereby effective or intergrain stress is reduced below its initial value, $(\sigma_i)_e$, with consequent loss of shear strength:

$$s = (\bar{\sigma}_{eff})_e \tan \phi \quad \text{for a cohesionless soil}$$

and might be reduced to zero if the excess pressure difference,

β , equals initial stress $(\sigma_i)_e$ due to overburden.

(3) While the full pressure difference, β , between the loads added at centerline and at toe, might not be transmitted laterally as excess pressure, nevertheless, the transmission of even a small proportion of the pressure difference, β , represents a dangerous condition by reducing the shear strength at toe of the embankment where the shear stress is relatively large. No method for quantitative analysis is known for analyzing this condition, but qualitatively Terzaghi has expressed the opinion that such lateral transmission of excess pressure may well have been the cause of several past embankment failures (4).

(4) The stability analyses covered above neglected such lateral pressure transmission and are therefore considered on the unsafe side, indicating a value of shear strength required in the silt which may be much too low. With many relatively pervious varves of fine

(4) K. Terzaghi (1943); "Stability of Fills above Horizontal Clay Strata", Proceedings of Sixth Texas Conference of Soil Mechanics and Foundation Engineering; University of Texas, August 1943.

sand in the Claremont silt, qualitatively there is considered to be good probability of a serious slide during construction unless measures are taken to minimize lateral transmission of excess pressure.

b. Inequality of strains to simultaneously develop shear strength in the dam and foundation is a further feature to be considered. The sand, gravel and glacial till in the dam embankment is much stronger than the soft foundation silt and reaches its ultimate strength at a much smaller strain. From a qualitative standpoint it is therefore not possible to develop full shear strength in both the dam and foundation at the same strain. If the full shear strength of the foundation silt is developed, the strains or deformations required are well apt to be sufficiently large as to cause small slips and cracks in the dam embankment deforming to the same amount of strain as the foundation silt. From a qualitative standpoint, therefore, it is very desirable that the safe strength of the silt be limited to a value appreciably below its ultimate and consistent with a small degree of strain more comparable to that required to develop ultimate shear strength in the strong materials of the dam.

c. From these features it is considered that the stability analyses covered above give results on unsafe side and that strains in the silt should be limited by keeping the stress appreciably below the ultimate strength. Even with the present flat slopes to reduce shear stresses and a construction period extended over three seasons to allow consolidation during gradual load application, the previous analyses are considered to qualitatively indicate very questionable stability.

d. The stability of the dam may be increased in three ways:

(1) Additional flattening of slopes and addition of berms, which is feasible but would be expensive.

(2) Further extension of the construction period in order to permit more consolidation to occur between periods of loading and thus increase shear strength of the silt. For reasons of river diversion and relocations, the construction period is now established at three seasons and any further extension is considered undesirable as well as uneconomical.

(3) Installation of drain wells in the soft silt foundation to furnish frequent outlets for escape of the excess pore water. The use of several lines of sand-filled drain wells extending from the pervious material above to the bottom of the soft silt and placed between the centerline and embankment toe will greatly reduce the amount of hydrostatic excess pressure transmitted laterally. By shortening the water escape path the rate of consolidation and consequent gain in shear strength, can be greatly accelerated by addition of drain wells - the time for consolidation varying inversely as the square of the distance the water must travel to a drainage face. Drain wells have the further advantage of economy and only slight interference with other construction operations.

D. FUNCTION OF DRAIN WELLS.

1. Proposed Installation.

a. The proposed drain well layout is shown on Plate No. A2 and includes approximately 240 sand-filled drain wells of 18-inch diameter placed over the entire foundation area except for a region under centerline of dam. Wells are omitted under this center region as there is less likelihood of this zone being included by a failure surface and since cut-off provisions are desirable here. The wells are spaced 60 feet on centers at the apexes of equilateral triangles to minimize overlap in zones of influence of adjacent wells. The drain wells extend entirely through the soft varved silt to compact soil below, except it is planned to omit wells at edges of the soft silt deposit where its thickness becomes less than 10 feet.

b. Drainage from the wells is collected at the top by a horizontal underdrain blanket of best pervious material placed 12 feet thick at base of the dam. While the alluvial layer of sand and gravel overlying the soft silt will serve to partly drain water from the wells, this deposit is not apt to furnish completely positive drainage due to frequent lenses of sandy silt. Therefore, the drainage blanket has been added at base of the dam to insure positive drainage.

2. Advantages of Drain Wells.

a. By furnishing frequent local outlets the drain wells serve to relieve hydrostatic excess pressure and minimize lateral transmission of these unbalanced pressures. Alternating spacing of drain wells in adjacent rows aids in intercepting the more pervious varves which account for the bulk of such lateral pressure transmission. In varved

soils the danger from lateral transmission of pore water excess pressure becomes greater as the ratio of horizontal to vertical permeability increases, and fortunately the same condition operates to increase the efficiency of drain wells in attracting radial drainage.

b. By reducing length of the water escape path, drain wells markedly accelerate consolidation which then occurs under the influence of both normal vertical flow and radial flow. Correspondingly, the rate of gain of shear strength is accelerated.

c. Acceleration in rate of settlement is also obtained but is more a by-product, which is desirable but not necessarily essential; as it is generally feasible to reliably estimate the ultimate settlement near end of the construction period from observations to that time and raise the height of dam accordingly.

3. Prior Use. - Sand-filled drain wells have been extensively used and developed by the California State Highway Department to accelerate consolidation in compressible foundation deposits. Observations of results have shown very marked increases in the rate of consolidation from a California experimental installation in 1934 and several subsequent applications to the foundations of highway embankments (5). It is also understood that several more recent installations of drain wells have been made in California on waterfront construction projects for war needs. Terzaghi (4) has mentioned an earlier application in approximately 1931

(4) Previous Reference.

(5) O. J. Porter (1936); "Studies of Fill Construction over Mud Flats Including a Description of Experimental Construction Using Vertical Sand Drains to Hasten Stabilization"; Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, Harvard University, Volume I, Page 229. Also covered in 1938 Proceedings of 18th Annual Meeting, Highway Research Board, Part II, Page 129.

to the foundations of a masonry dam on the Swir River in Russia, but it is not clear if the main purpose of this installation was to accelerate consolidation. In 1941-42 the Providence District installed a system of drain wells through varved clay in repairing a slide at Riverfront Dike, Hartford, Connecticut.

4. Theory. - The theory of consolidation by combined vertical and radial drainage has been developed independently by L. Rendulic (6) and N. Carrillo (7) both working under the direction of K. Terzaghi, and by R. A. Barron (1) of the Providence District Soils Laboratory. In this analysis of design, additional solutions by Barron are given to cover effects of well resistance and smear at periphery of the well.

-
- (1) Previous Reference.
 - (6) L. Rendulic, (1935), Der hydrodynamische Spannungsausgleich in zentral entwässerten Tonzylindern, Wasserwirtsch.u.Technik, Vol. 2, p. 250-253, 269-273.
 - (7) N. Carrillo, (1941), "Consolidation of a Soil Stratum Drained by Wells", mimeographed by Harvard University, Cambridge, Mass. Also covered in Journal of Mathematics and Physics, Volume XXI, No. 1, March 1942.

E. STABILITY ANALYSIS WITH IDEAL DRAIN WELLS.

1. Case Considered. - Initial studies of effect of drain wells

on stability of dam and foundation were made considering ideal drain wells, 12 inches in diameter, having no resistance to flow up the wells and having no remolded or smeared zone adjacent to the well. The final design differs slightly with well diameters tentatively increased from 12 to 18 inches. Because of large amount of work required to revise extensive computations based upon use of 12 inch wells, the analysis in this appendix considers only 12 inch diameter wells.

2. Analysis Method.

a. The assumptions used in studying the effect of drain wells were the same as those given in Paragraph C.2, except that both normal vertical flow and radial flow to wells were considered. The ratio of horizontal to vertical permeability (k_H/k_V) was assumed to be 10 as a very conservative choice. It was also assumed that only vertical strains resulted from consolidation.

b. The method of analysis was quite similar to that described in Paragraph C.3, Page 14, except that the combined effects of seepage radially to drain wells and vertically to upper drainage face was considered.

(1) The hydrostatic excess pressure at any depth "z" and for time "t" due to a load " u_0 " added instantaneously is obtained by the formula

$$u'_z = u_z \frac{\bar{u}_r}{u_0}$$

Where " u_z " is the hydrostatic excess pressure at depth z. for consolidation

by vertical flow only to a single drainage face and \bar{u}_r is the average hydrostatic excess pressure in the zone of influence of each drain well for consolidation by radial flow only to the well. Therefore, " u'_z " is an average value of hydrostatic excess pressure at depth z for consolidation by combined flow - an average for the zone of influence around each drain well.

c. To permit the construction of contours of hydrostatic excess pressure, u'_z , for case of consolidation by combined vertical and radial flow, curves M and B of Plate No. A27 were plotted to give the percent excess pressures at the bottom and mid-depth of silt deposit in terms of time factor " T_v ". Values are percentages of initial hydrostatic excess pressure, u_o , which is equal to added load. These curves apply to any depth of silt, " H ", and were obtained from curves on Plate No. A28, which is for the conventional case of consolidation by vertical flow and shows relation of consolidation with depth for any time t , where $t = f(T_v)$

Shown also on Plate No. A27 is a curve R for average percent hydrostatic excess pressures for time factor " T_H " for the condition of consolidation by radial flow to a drain well only, with $n = 80$. The values for this curve were obtained from Plate No. A29, which is for radial flow only and gives excess pressures as averages on any horizontal plane within the zone of influence of a drain well. Both Plates Nos. A28 and A29 were taken from District Publication on Drain Wells (1).

(1) Previous Reference.

(1) In these curves the time factors T_v and T_H are dimensionless numbers and are related to time t as follows:

For consolidation by vertical drainage only

$$T_v = \frac{k_v (1+e) t}{\gamma_o a_v H^2}$$

For consolidation by radial drainage only

$$T_H = \frac{k_H (1+e) t}{\gamma_o a_v d_o^2}$$

The factor n , drain well-diameter ratio, is defined as

$$n = \frac{\text{diameter of Influence Zone}}{\text{well diameter}} = \frac{d_o}{d_w}$$

In the above expressions γ_o is the unit weight of water and a_v is the coefficient of compressibility.

(2) For consolidation by vertical flow to a single upper drainage face, the data for plotting curve M, Plate No. A27, were obtained for different values of T_v at mid-depth of chart on Plate No. A28. Curve B, Plate No. A27, for bottom of silt was obtained in similar manner from base of chart, Plate No. A28.

(3) For consolidation by radial drainage only to 12 inch diameter drain wells spaced 80 feet apart, $n = 80$, and for this value of n the percent average hydrostatic excess pressures were taken off Plate No. A29 for given values of T_H and plotted on Plate No. A27 to obtain curve R. The average excess pressure, which is independent of depth z , is used because any possible failure surface to be investigated later would cut across the entire zone of influence of each drain well.

(4) The values of hydrostatic excess pressure

curves on Plates Nos. A27, A28, and A29 are percentage values for instantaneous loadings. The true value of pressure is obtained by multiplying the real load by these percentages.

d. The curves on Plate No. A27 are independent of actual thickness, H , of the compressible soil deposit and to be of value must be converted for a finite thickness and time factors converted to time.

(1) For a soil deposit 56 feet thick having an average void ratio $e = 0.95$ and a coefficient of consolidation, C_v , of 35×10^{-4} cm²/sec. the relationship between time, t_{months} , and T_v may be found as follows:

$$t = \frac{\left(\frac{H}{1+e} \right)^2 T_v}{C_v}$$

And including proper conversion factors

$$t_m = \frac{\left(\frac{56}{1+0.95} \right)^2 T_v}{(35 \times 10^{-4}) \frac{60 \times 60 \times 24 \times 30}{30.5 \times 35.5}} = 84.5 T_v$$

The percent hydrostatic excess pressure for the mid-depth and bottom from Plate No. A27 were then plotted in Diagram VI, Plate No. A26, curves Y and X, against time, t_m , using the above relation.

(2) If 1 foot diameter drain wells are installed in the above deposit with a spacing of 80 feet then " n " = $\frac{d_e}{d_w} = \frac{80}{1} = 80$.

For a k_H/k_v ratio of 10 and making use of the definitions of T_H and T_v as given on Plate No. A27, the time factors, T_v and T_H , for any given time are then related as follows:

$$T_v = \frac{T_H d_e^2 k_v}{H^2 k_H} = \frac{T_H 80^2}{56^2} \times \frac{1}{10} = 0.204 T_H$$

and $t_{\text{months}} = 17.2 T_H$

(3) The average percent hydrostatic excess pressure at any time t_{months} for case of combined radial and vertical drainage may then be found for given depth by above formulae using curves as given on Plate No. A27 or interpolated from Plates Nos. A28 and A29. As an example, to find the average percent hydrostatic excess pressure at the bottom of the layer for the conditions at $t_m = 10$ months:

$$T_H = \frac{10}{17.2} = 0.58,$$

$$T_V = \frac{10}{84.5} = 0.118$$

For $T_H = 0.58$, \bar{u}_r from radial drainage curve on Plate No. A27 is 30% and for $T_V = 0.118$, u_z for base of layer, also from Plate No. A27, is 92%.

$$u'_z\% = 30 \times \frac{92}{100} = 27\%$$

In this manner curve Z was constructed as shown on diagram VI, Plate No. A26.

(4) For the case of instantaneous loading the drain wells have reduced the average hydrostatic excess pressure at bottom from 92% for single vertical drainage to 27% and at the mid-depth of the layer from 70% to 20%. Thus the wells have displaced curve X on Plate No. A26 to curve Z and curve Y to W indicating the great benefit of wells in accelerating consolidation. If the k_H/k_V ratio is greater than $\alpha = 10$ that was used above, the effect would be even more marked.

c. Effect of gradual loading was accounted for as shown on Plate No. A26. In diagram V the upper broken line is the loading curve. At the end of the first season's work the average load is 3.3 Tons/sq.ft. while at the end of the second season it is 6.0 Tons/sq.ft.

The time scales for diagrams V and VI have no relationship except that the units are equal.

(1) The variations of hydrostatic excess pressures with time as the load is added are shown on diagram V of Plate No. A26. To illustrate how these curves were obtained, a point A at end of 18 months will be determined as shown on diagram V for the case without drain wells.

(2) At time t_a the load is l_a tons/sq.ft. and a small increment of load Δl_a is placed which remains as a consolidating load for a period of $(18-t_a)$ months. The percentage of initial hydrostatic excess pressure at the end of $(18-t_a)$ months is $u_z\% = 85\%$ from curve X. This value is then plotted in diagram III above the load value of l_a . Other points may be found for other times "t" less than 18 months and plotted in diagram, including that from load increment applied at 18 months, which has been on only for an instant and therefore develops 100% of initial hydrostatic excess. A curve may then be drawn through these points as shown.

(3) Now $u_z\% \times \Delta l_a = \Delta u$, the hydrostatic excess pressure in tons/sq.ft. at 18 months. Therefore, the area under the curve is the total hydrostatic excess pressure at the bottom of the layer at 18 months which is consolidating by vertical flow only. The value of this area by planimeter is 13.45 sq. in. while that for the entire rectangle is 15 sq. in. The pressure, u , then is

$$\frac{13.45}{15} \times 6 = 5.38 \text{ Tons/sq.ft.}$$

(4) Shown also on diagram III are curves and values for other conditions. Diagrams I, II, III and IV cover determination of

hydrostatic excess pressures at different times thus permitting the construction of the hydrostatic excess pressure - time curves as shown in diagram V. The case with drain wells is handled in the same manner by using curves W and Z of diagram VI.

f. To permit the construction of hydrostatic excess pressure contours as shown on Plate No. A25, the procedure of Plate No. A26 was performed at a number of points in the compressible silt deposits at mid-depth and bottom for case of consolidation by vertical flow only and also by combined vertical and radial flow. As illustrated on Plate No. A25, these hydrostatic excess pressure curves show a marked decrease of pressure when wells are provided as compared to the case with no wells.

3. Analysis Results.

a. Downstream Slope. - For a safety factor of 1.00 and neglecting any minor amount of cohesion present in the soft varved silt, the required shearing angles for different well spacings studied are as follows:

	<u>50 ft. spacing</u>		<u>80 ft. spacing</u>
End of 1st construction season	10°	(13°)	11-1/4° (14-1/4°)
End of 2nd construction season	10°	(13°)	10-1/2° (13-1/2°)
End of 3rd construction season	11-1/2°	(14-1/2°)	11-1/2° (14-1/2°)

A value of 3° has been added for same reasons as given in Paragraph C.4.b. (Page 17). Results indicate for the spacings used that the difference in effects are not large. This is indicated, as regards acceleration of consolidation by drain wells, on Plate No. A31. The factor "R" may be considered as a reciprocal of an efficiency factor; the lower the values

of R the more effective are the drain wells. It should be noted, however, that these values of R are for instantaneous loadings.

b. Upstream Slope - Sudden Drawdown. - The stability of the upstream slope was checked for case of a sudden drawdown of the reservoir pool at end of construction season. This is very severe as it assumes that pool is filled during 3rd construction season and emptied instantaneously at the end of the construction so that the load on the foundation is a maximum. For case of 12 inch diameter drain wells spaced 80 feet on centers the required friction angle for a stability factor of 1.00 is 15° which becomes 18° when the 3° are added as previously discussed.

c. Closure Section. - It is planned to divert the Sugar River through the outlet works at the start of the 3rd construction season and to complete the dam embankment in this season by constructing the required closure section as shown on Plate No. A5. Although the dam, at this location, will be raised to its full height in one season, the section is not so critical because:

- (1) The foundation silt is much thinner.
- (2) Section is short with a strong foundation (till and rock) on the left abutment and with the foundation on the right side of section strengthened as pre-loaded by the dam for 1st and 2nd seasons.

Any potential slide in this area would have a 3-dimensional aspect; however, for purposes of simplification the stability analysis assumed a 2-dimensional case which is more severe. The required friction angle for safety factor of 1.00 is 10° which becomes 13° when the 3° are added.

1. EFFECT OF WELL RESISTANCE.

1. Well Resistance. - The above studies were based upon ideal wells of infinite permeability, offering no resistance to flow in the well. Actually, head losses will occur due to resistance to flow offered by the sand backfill in the well and this somewhat reduces the rate of consolidation. If the flow is large or if the well cross sectional area is small, then the back pressure due to well resistance will be high. On the other hand, if the flow is small, as for shallow deposits or for very tight soils, or if the well area is large, then the resistance of the well to flow will be small. A very pervious filling would be ideal from the standpoint of minimizing well resistance; but filter action is needed also with sufficiently small voids in the sand filling to prevent inwash of the surrounding soil.

2. Well Backfill. - The proposed well backfill material is a medium sand of uniform gradation as shown on Plate No. A32. This material will be obtained from Borrow Area "A" by selection and placed in the wells at a loose density. For this condition the permeability will range from 100×10^{-4} to 300×10^{-4} cm/sec., average value probably about 200×10^{-4} cm/sec.

a. This sand backfill will serve as a filter for the fine sand and coarse silt layers of the soft varved silt. A filter test run on a typical sample of remolded silt (see Plate No. A32) was stable when the filter material was on the fine side of the proposed gradation range of well backfill and not quite perfectly stable for filter material from the coarse side. These tests were all run under a very high gradient ($i = 3000+$) which is very much greater than that expected in the field

and was employed to counteract the comparatively short time of the test.

b. It is not considered practical to obtain a positive guarantee of perfect filter action; as, if finer well backfill were used, objectionable well resistance would result. The proposed sand backfill is considered an adequate filter for the varves of fine sand and coarse silt, through which bulk of flow will occur. The backfill is not a perfect filter for the varves of lean clay and finest silt; but very little flow tending to move soil grains will occur in these soils and the small cohesion present will considerably resist this, particularly for the lean clay.

(1) Recently an undisturbed sample was obtained from another bed of varved soils where a layer of clean, Class 4-2 very uniform, coarse sand about 0.07 feet thick was found between two layers of Classes 8 and 10 silt and lean clay. Examination showed the sand to be very clean and free of fines infiltrated from the adjacent soils even though this soil has probably been subject to many reversals of seepage direction in the past several thousand years. The absence of infiltration here is evidence that the gradients in nature were not sufficient to move the fine silt into the comparatively large voids of the adjacent coarse sand.

3. Mathematical Solution. - A solution for effect of well resistance for the case of uniform vertical strain at any depth "z" has been obtained by Mr. R. A. Barron and is shown on Plate No. A33. For case of simplicity, flow in a vertical direction has been neglected and the solution covers a case where both upper and lower boundaries of the compressible soil are impervious. Therefore, the solution found is slightly slower than

true condition. However, as the ratio of k_H/k_v becomes larger this difference becomes smaller.

a. Although not strictly correct from a physical point of view, an approximate solution can be obtained for case of consolidation by both radial and vertical flow with well resistance as follows:

$$\bar{u}_{z,r} = u_z \cdot \frac{\bar{u}_r}{u_0}$$

Where " $\bar{u}_{z,r}$ " is the average hydrostatic excess pressure at depth "z", " u_z " is hydrostatic excess pressure at depth z for case of consolidation by vertical single drainage and \bar{u}_r is average hydrostatic excess pressure at depth "z" as given by equation 5 on Plate No. A33. The term u_0 is the initial hydrostatic excess pressure at time zero and is equal to the applied load. The total overall average excess pressure is given by $\bar{u} = \bar{u}_z \times \frac{\bar{u}_r}{u_0}$ where \bar{u}_z is the average pressure for consolidation by single vertical drainage and \bar{u}_r is obtained from equation 8, Plate No. A33.

(1) A study of the total overall hydrostatic excess pressure at downstream slope of dam for various spacing of wells and k_H/k_v ratios is summarized on Plate No. A34. The effect of well resistance is shown on this plate by the curves for 80 foot well spacing, with and without well resistance considered.

G. EFFECT OF SMEAR.

1. Description of Smear. - If drain wells are installed by cased holes and then backfilled as casing is withdrawn, the driving and pulling of the casing will distort and remold the adjacent soil. In varved soils the finer and more impervious layers will be dragged down over the more pervious layers resulting in a zone of reduced permeability immediately adjacent to the drain well. An example of this distortion is shown on Plate No. A30 showing some smear from an early type of undisturbed soil sampler (M.I.T. spoon with 5-inch tubing and 1/4-inch wall) in a soil with relatively thick individual varves. It is not difficult to visualize a far greater smear from the use of a heavy casing or hollow mandrel in drain well construction. Such would be apt to become more pronounced where individual varves are comparatively thin, as at Claremont.

a. The California practice has been to use casing or hollow mandrel to install wells. The soils, however, have been mainly peat, harbor silt, and other similar soils which are probably not particularly stratified. Although remolding by the casing will reduce the permeability of the surrounding soil, the reduction should not be anywhere near as serious as that for varved soils.

2. Mathematical Solutions.

a. An approximate indication of the retarding effect of a zone of reduced permeability at well periphery, may be had for a constant rate of radial flow from a solution by Muskat (8). During study of

(8) M. Muskat (1937); "The Flow of Homogeneous Fluids Through Porous Media"; McGraw-Hill Book Co. P.403,

Claremont foundation problems, R. A. Barron of the District Soils Laboratory obtained two solutions for the case of consolidation by radial flow with a zone of reduced permeability adjacent to the drain well. These solutions differ from Muskat's, in that the rate of radial flow varies as consolidation progresses and is not constant. Solution No. 1, (see Plate No. A35), is for a case where vertical strains at any depth are not equal and solution No. 2 (see Plate No. A36) is for a case where such strains are all equal.

b. The principal difficulties in using these solutions are the selection of an average permeability, k_s , of the smeared or remolded zone and the extent of this zone. For stratified soils, such as varved silts, it would appear that " k_s " would be of the order of magnitude of k_v , the average permeability in a vertical direction. The extent of the remolded or smeared zone will depend upon the well installation method:

(1) If the well is installed by careful augering and jetting, the smearing may be little or none.

(2) If the well is installed by driving a casing which is cleaned out as it is sunk and then pulled as well as backfilled, the remolding or smear may be of fair magnitude, especially if externally flush jointed casing is not used.

(3) If a hollow mandrel is driven with a detachable point so as not to require cleaning, then the soil will not only be remolded by friction of the casing but also by being displaced and consequently the remolded zone will probably be of a very substantial thickness.

c. Although some difference exists between the initial hydrostatic excess pressure distributions for the above noted solutions

for uniform loads instantaneously applied, as consolidation progresses this difference diminishes. The average hydrostatic excess pressures, however, are always very near equal. Inasmuch as the equal strain case is much easier to handle, the following examples are based on this solution.

(1) Assume the well diameter-zone of influence ratio, $d_e/d_w = n = 60$, and the ratio of smeared zone to well diameter, $r_s/r_w = m = 1.5$ (for a 12-inch diameter well the smeared zone thickness, $r_s - r_w$, is 6 inches). Then the average hydrostatic excess in percentage of initial load is $\bar{u}/u_0 = 100e^{-\frac{8T_H}{F(m)}} = 100e^{-\frac{8\omega tk_H}{10 F(m)}}$ where T_H is a time factor and $F(m)$ is defined on Plate No. A36. The term ω is a constant depending on physical properties and dimension so that $\omega tk_H/10 = T_H$ where "t" is time.

(2) Now for different ratios of k_v/k_H and smear:

$k_H/k_v = 10$		$k_H/k_v = 100$	
$m = 1.0$	$m = 1.5$	$m = 1.0$	$m = 1.5$
no smear	smear	no smear	smear
$\bar{u}/u_0 : 100e^{-2.39 \omega t_1}$	$100e^{-1.14 \omega t_2}$	$100e^{-23.9 \omega t_3}$	$100e^{-1.84 \omega t_4}$
Now for any value of \bar{u} , the t to reach that percent of consolidation in terms of t_1 is			
$1.00 t_1$	$\frac{2.39 t_1}{1.14} = 2.10 t_1$	$\frac{2.39 t_1}{23.9} = 0.10 t_1$	$\frac{2.39 t_1}{1.84} = 1.30 t_1$
			$\frac{2.39 t_3}{1.84} = 13.0 t_3$

Therefore, it is apparent that for equal smear zones having $k_s = k_v$, the soil having a permeability ratio of $k_H/k_v = 100$ will consolidate faster

than a soil having a similar ratio of only 10. However, the reduction in consolidation rate because of smear is much greater for k_H/k_v equal to 100 (13 times as slow) as compared to that for $k_H/k_v = 10$ (2.1 times as slow).

d. A solution has also been obtained for the case of equal strain at any depth z for consolidation by radial flow with both smear and well resistance present (see Plate No. A36). For case of instantaneous loading, consolidation curves are shown on Plate No. A37 for various cases of smear conditions with and without well resistance. Case for $m = 7/6$ (1-inch thick smear for 12-inch well) would appear to be a minimum for installation of wells using a casing, while $m = 3$ (12-inch thick smear for 12-inch well) would appear to be nearly a maximum for a hollow mandrel driven with total displacement of soil. For the first case the lag caused by well resistance is about equal to that due to smear; whereas in the second case the well effect is much the smaller of the two.

3. Importance of Smear Reduction. - From the above discussion it is apparent that it is very important to minimize smear at wells in varved soils, to obtain the maximum benefits from drain wells. During the installations of drain wells at Hartford, Connecticut in 1942 by the Providence District, the importance of minimizing smear in varved soils was recognized. The wells were sunk uncased through the varved clay and clay was excavated by a special cutting auger in combination with jetting action of water.

a. With advice of the engineering consultants for Claremont Dam, it is proposed to install experimental wells in advance of advertisement of bids to determine the relative efficiency of the two

following proposed methods of well installation.

(1) Install wells by driving an 18-inch casing, cleaning out casing as it is advanced. Then backfilling casing with sand as casing is removed.

(2) To minimize smear install wells uncased by use of special auger cutting by water jets, which is now being designed by District exploration forces.

(3) Wells will be tested by lowering the water in them and then observing rate of inflow.

b. In the stability analyses the effect of smear has been neglected in belief that it can be kept to a minimum by method (2) above and because it is believed that the use in the studies of $k_H/k_v = 10$ is very conservative.

H. WELL STIFFNESS

1. Deformation of Wells.

a. Although sand backfill in the drain well is placed in a quite loose state, it is reasonably certain to be less compressible than the surrounding varved silt. From this the drain well has some tendency to act as a sand pile or hard point in the foundation, attracting load and somewhat relieving stresses in the surrounding compressible silt deposit. A solution considering a definite distribution of load between the well as a sand pile and the remainder of the foundation has not seemed feasible at present due to the uncertainty in assumptions necessary. However, a limiting case has been solved as covered in the previously mentioned Providence District Bulletin (1). From this it appears that the influence of drain wells attracting load has very little effect on the overall rate of consolidation of the compressible deposit, although the stress disposition on any plane in the soil is affected considerably (see Figure 34 of reference (1)).

b. Of more concern is action of the drain well in deforming under this load concentration as the surrounding compressible soil settles - up to a maximum of 3-1/2 feet estimated at this site. Deformation of the well may be accomplished either by bulging of the well column of sand with small increase in density, or by local shearing with possibly a relative displacement of the sand column.

(1) Bulging is not objectionable and for a maximum settlement of 3-1/2 feet at Claremont Dam, drain wells 80 feet long would have to increase uniformly in diameter only the following small amounts:

12 in. diameter well to 12.3 in. diameter
18 in. diameter well to 18.4 in. diameter
24 in. diameter well to 24.5 in. diameter

(1) Previous reference.

The increase in diameter would be less than this if allowance is made for densification of the sand backfill from its initially loose state.

(2) A definite shearing of the well column could be very undesirable if continuity of the drainage path in the well were to become broken by intrusion of impervious silt or lean clay from the surrounding deposit. If an intrusion of substantial thickness (6 to 12 inches) were to interrupt the drainage path, it might even nullify the effect on consolidation contributed by portion of the well below the broken zone.

2. Past Experience.

a. The effect of well deformation under attracted load was considered in the 1941-42 Providence District installation at Hartford, Connecticut. At the suggestion of Dr. A. Casagrande, consultant, attempt was made to install cushion layers of cinders, which material is more compressible than the sand backfill and substantially as pervious. Purpose of the cushion layers in the well backfill was to increase the well compressibility and reduce chances of sudden shear failure in the well column, as there was some concern that a shock accompanying shear failure might cause a temporary increase in the hydrostatic excess pressure in the disturbed varved clay below this slide area. However, the cinders obtainable prove to be a poor filter against intrusion of the silt strata in the surrounding varved clay and difficult to install, tending to sink through water slowly and to mix with the sand as well as break down in grain size thus reducing permeability. Accordingly, the use of cinder cushion layers was abandoned and bulk of the drain wells backfilled entirely with sand.

(1) In the absence of borings at the sides of the completed wells, no positive evidence is available whether these Hartford

drain wells deformed by bulging or by shearing; but at least no ill effects were observed.

b. The California Highway Department has employed drain wells from 20 to 30 inches in diameter on 10 to 20-foot centers beneath fills where the observed settlement has totaled several feet. More recently, it is understood the California practice has included 12 to 18-inch diameter wells up to 75 feet long where the observed settlement has been 4 feet and larger. From conversations with Mr. O. J. Porter and Mr. T. E. Stanton, Jr., of the California Highway Department, it is understood that their several installations have been very successful; and if any damage to the drain wells did occur by shearing, it was of such minor nature as to escape notice in the records obtained of the consolidation rate actually occurring.

3. Probable Well Action. - The possibility of shearing of the well column of sand reducing efficiency of the drain wells is more probably confined to their use in stiffer clays. In the soft silt at Claremont Dam, deformation by bulging is believed more probable, considering that the soft silt needs to be compressed laterally only about 1/4 inch in each well to accommodate the maximum 3-1/2-foot settlement.

I. SELECTION OF WELL SPACING AND DIAMETERS

1. General. - The stability analysis discussed in Section E of this appendix is based upon the use of 12 inch diameter ideal drain wells spaced at apexes of equilateral triangles 80 feet apart. A value of k_H/k_v of 10 was used, and it was assumed that no well resistance or smear was present to reduce consolidation rate. Plate No. A34 indicates the hydrostatic excess pressures for the first season construction for well spacings of 80, 60, and 50 feet corrected for well resistance for cases of k_H/k_v equal to 10 and 50, respectively. Values were computed for the 80 and 50 foot spacings and interpolated for the 60-foot spacing. Also shown on Plate A34 is a curve for consolidation by vertical flow only (without wells) and another for ideal well at 80-foot spacing as used in the stability analyses noted above.

2. Favorable Features. - Favorable features of the conditions analyzed are mainly that the soil constants used are considered quite conservative.

a. The ratio $k_H/k_v = 10$ used in the stability analyses is probably quite low. Although no determinations of this ratio were made for Claremont silts, subsequent tests on similar silts showed ratios varying from about 5 to 80, with probability that the ratio is higher in nature.

b. The permeability of the drain well backfill will probably range from 100×10^{-4} to 300×10^{-4} cm/sec. For study purposes a value of 200×10^{-4} cm/sec was used and is considered conservative.

c. The studies of effect of well resistance have been for a section where silt is the deepest and therefore the amount of water to be discharged through the well is the greatest. At other locations where silt deposit is thinner, the wells will have to handle only smaller rates of flow, which will result in well resistance contributing less effect on the

consolidation rate.

3. Unfavorable Features.

a. Smear was not considered in the stability analyses, but may occur even with the use of methods proposed for minimizing this feature.

b. Revised construction rate, as determined after completion of major portion of stability study, is shown on Plate No. A38, indicating that the first season's load will be somewhat less and the second season's load somewhat greater than in the original rate considered in stability analysis. However, as shown on Plate No. A38 the hydrostatic excess pressure at the end of the second season for the new rate, using 12 inch diameter wells at 60-foot spacing and considering well resistance, is somewhat less than that for 80-foot spacing of same diameter ideal wells. To account for some reduction in well area due to filter action of well backfill not being quite perfect, a reduction of diameter has been assumed for the 60-foot spacing from 12 to 10 inches, and this considered in computing the curve on Plate No. A38.

c. Some of the drain wells may shear off during the process of foundation settlement but based upon past experience of this District at Hartford and the more extensive experience of the California State Highway Department it is expected that this item will be only of a minor nature.

4. General Considerations.

a. A 50-foot spacing of wells is satisfactory but probably unnecessarily close. The estimated number of wells for this spacing is 425. A spacing of 80 feet (175 wells required) is considered too wide, considering well resistance and possibility of some minor smearing. A spacing of 60 feet (240 wells required) was chosen as reasonable selection based upon indications of Plate No. A34. Some minor smear that may develop

will probably be offset by the true value of k_H/k_V being much greater than the ratio of 10 used in the computations.

b. The later increase in diameter of the drain wells, from 12 to 18 inches, will slightly reduce the effects of well resistance, permit a minor amount of smear for same rate as a 12 inch well with no smear, and also provide for loss of well area from imperfect filter action of well backfill without any serious increase of well resistance. For these reasons the board of consultants tentatively suggested increase in diameter to 18 inches, final selection to be governed by data from installation of experimental wells. These test wells, to be installed in the spring or summer of 1945, will be used to obtain data to determine final minimum diameter and type of installation method.

c. If the experimental drain wells indicate smear and if that cannot be offset by increasing the diameter of wells to 18 inches, then the proposed 60-foot well spacing may have to be reduced somewhat. On the other hand, if the results of the experimental wells are very favorable and construction methods are feasible, then there may be a possibility that 12 inch diameter drain wells can be used.

J. DRAIN WELL CONSTRUCTION METHODS.

In the original analyses 12-inch diameter wells were considered as it was initially estimated this size was most economically suited for construction equipment readily available. In the final design, well diameter has been tentatively increased to 18 inches for reasons previously discussed with the expectation that cost would not be unduly increased. Experimental wells are planned to investigate construction methods for reducing smear and to study the effect of diameter from which field experiments, final construction methods and well diameter will be selected, as well as any changes in well spacing if such seems necessary. Details and extent of this experimental program have not yet been fully determined but the following is a preliminary outline.

1. Two construction methods are planned for installing these experimental wells.

- a. A method is being actively considered to minimize smear by sinking the well uncased in the silt and excavating by action of water jets of a special cutting auger (9). The method proposes to case through the 15 to 20 feet of sand and gravel deposits overlying the silt. This casing will be cleaned out by jetting and any large cobbles remaining will be pushed aside by the use of a special spud. The drain well will then be advanced as an uncased hole in the silt using the special cutting auger combined with jetting to aid cutting and removal of soil from hole. When the auger reaches the bottom of the soft silt deposit, a mixture of sand and water will be pumped in by a pump-crete machine to the bottom of the well through a water supply pipe to the auger. As the sand backfill is deposited the auger will be removed.

(9) Devised by F. E. Fahlquist - Head of Foundation Investigations Section, Providence District.

The rate of auger removal will be adjusted so that it is just below the top of placed backfill. With a constant flow of water and the maintenance of an excess water head inside the well, it is not expected that the well will close up before sand is placed. However, if this occurs the auger will be able to clean the hole out as it is removed depositing the sand below the auger.

b. A second method of installing drain wells will be to drive an open end, flush-jointed casing to bottom of the soft silt. To prevent excess remolding of soil as the casing is being driven, it will be kept cleaned out either by jetting or by use of compressed air. Upon reaching the bottom of the soft silt the casing will be cleaned out and backfilled with sand. As the sand backfill is being placed the casing will be withdrawn, keeping the top of the sand fill just above the bottom of the casing to prevent silt from squeezing off the backfill. With this more conventional method of well installation a considerable amount of smear is expected at the well periphery caused by driving and removing of casing.

2. It is planned to further investigate effects of construction method on the amount of smear by sinking shallow holes with the above two methods in silts similar to those at Claremont which are found above the water table at several places in the Connecticut River Valley. By working above the water table it will be possible to sink shallow drain wells and then expose the result in a test pit to observe the amount of smear created and to secure undisturbed samples for testing. To partly simulate conditions at Claremont where the silt is well below the water table, it is probable the test area will be partly saturated by surface ponding for a considerable period before the tests.

3. It is also planned to place well points or similar pump screens in the backfill of the experimental wells and to conduct pumping tests to

furnish some data on the effect of smear from the different construction methods. The wells will be tested for infiltration capacity by lowering or raising the water table within the well backfill and by observing the rate of change in the wells and at piezometers placed around the wells. During such tests the sand and gravel overlying the soft silt will be sealed off by casing.

4. The present layout of drain wells on Plate No. A2 is based on the probable boundaries of the soft silt deposit as determined from available borings. During installation of the wells a record will be kept of the extent of the silt deposit actually encountered and revisions to the well layout made as necessary in the edge zones of the silt deposit.

K. OBSERVATION DEVICES.

1. Piezometers. - Because of the magnitude of Claremont Dam and the soft, varved silt deposit in the foundation which is highly stratified, it is very important that the hydrostatic excess pressures be kept within allowable limits and distribution at all times. An extensive piezometer layout will, therefore, be installed in the soft varved silt at strategic locations, and observations frequently taken to record actual hydrostatic excess pressures and their rates of change. These piezometers will serve as a warning system in event drain wells do not function as well as analyzed or load is added so rapidly as to create unsafe conditions.

a. Because of the time lag that occurs with open pipe piezometer in very impervious compressible soils, consideration is being given to the possible use of the electric strain gage type of piezometer which requires only negligible movement of water between soil and piezometer tip in order to reflect pressure changes. However, at present the exact design and location of these piezometers have been deferred until experience of others with this type of gage has been determined.

2. Settlement Gages. - Settlement gages will be installed on the dam foundation and carried up as the embankment progresses to obtain a record of settlement. To remove the possibility of the fill gripping the gage pipe and pushing it into the foundation as the embankment compacts under its own weight, a slip joint is provided just above the foundation. The lower part of the gages will be perforated and protected from infiltration of soil by fine mesh screen and sand filter layer to permit observations on water table. When pool is low the water table at the downstream portion of dam may be somewhat below the top of foundation. Therefore, these gages have been equipped with well point extensions to present minimum ground water elevation.

3. Lateral Movement. - Monuments are planned along the toes of the embankment and also possibly on the embankment slope to observe any lateral movement. These will be used in conjunction with the piezometers to detect any overstressing of the foundation to prevent foundation failures. Monuments will be set so as to be free of any danger of frost heaving.

4. In accordance with policy for expediting general designs, the main design plans and specifications for Claremont Dam have been completed except certain phases requiring further study and development (type and location of piezometers and monuments) have been deferred for the present. The settlement gages will be installed by the embankment contractor while the other observation devices will be installed by government forces.

L. RELIEF WELLS.

1. Deeply Buried Pervious Deposit. - As shown on Plate No. A7, a deposit of pervious sands deeply buried by impervious overburden, exists in the bottom of the pre-glacial bedrock valley extending from some point above the dam site to at least 3000 feet downstream. The existence and location of any inlet or outlet to the ground surface for this deposit is not known from present borings. As a rock barrier rises to the surface in the present river valley about one mile downstream in the City of Claremont, there is at least a possibility that the pre-glacial outlet of this buried pervious is now blocked by impervious glacial till.

2. Uplift Pressures - No Wells. - If it is assumed that an outlet for this deposit does not exist and that an inlet does exist within the reservoir area, it is possible that excessive water pressures may develop within the deposit at downstream toe of the dam.

a. Using the symbols and formulas shown on Plate No. A39, an estimate of this uplift pressure may be closely approximated for reservoir at spillway, assuming an inlet at 10,000 feet above upstream toe of dam and a barrier 3000 feet downstream blocking any outlet.

$$L_1 = 10,000 \text{ ft.}$$

$$L_2 = L_1 \div 1000' \text{ (dam width)} \div 3000' \text{ (length of pervious below dam)}$$

$$L_2 = 14,000 \text{ ft.}$$

$$X = 11,000 \text{ ft. (downstream toe of dam)}$$

$$A_1 = 100 \text{ ft.}$$

$$A_2 = 40 \text{ ft.}$$

$$k_1 = 0.001 \times 10^{-4} \text{ cm/sec}$$

$$k_2 = 50 \times 10^{-4} \text{ cm/sec}$$

$$H = 105 \text{ ft.}$$

Then from the equations of Plate No. A39

$$c = \frac{k_1}{\sqrt{k_2 A_1 A_2}} = \frac{\sqrt{10 \times 10^4}}{\sqrt{50 \times 10^8 \times 100 \times 40}} = 0.707 \times 10^{-4} \frac{1}{\text{ft.}}$$

$$cL_1 = 0.707 \times 10^{-4} \times 10^4 = 0.707$$

$$c(2L_2 - L_1) = 0.707 \times 10^{-4} \times 1.8 \times 10^4 = 1.272$$

$$cX = 0.707 \times 10^{-4} \times 1.1 \times 10^4 = 0.778$$

$$c(2L - X) = 0.707 \times 10^{-4} \times 1.7 \times 10^4 = 1.202$$

$$e^{cX} = e^{0.778} = 2.178$$

$$e^{c(2L - X)} = e^{1.202} = 3.33$$

$$e^{cL_1} = e^{0.707} = 2.030$$

$$e^{c(2L - L_1)} = e^{1.272} = 3.58$$

$$\tanh cL_1 = \tanh 0.707 = 0.6088$$

$$1 - \tanh cL_1 = .3912$$

$$1 / \tanh cL_1 = 1.6088$$

$$a_1 = \frac{105}{.3912 \times 2.03 / 1.6088 \times 3.58} = 16.03$$

$h_x = 16.03 (2.178 / 3.33) = 88\text{-}1/2$ ft. in excess of ground water, as uplift pressure on base of impervious overlying the pervious deposit at the downstream toe of dam.

b. If the underground pervious deposit has a much wider area or higher permeability upstream from the dam, this excess head may be somewhat higher possibly of the order of 95 to 100 feet. Assuming such condition with an excess head of 97 feet, safety against flotation at downstream toe is analyzed as follows. The total overburden above the buried deposit will have a vertical pressure of about 4.1 tons/sq. ft. computed from moist soil weight above water table and buoyant below. The factor of safety against uplift then is:

$$F.S. = \frac{\text{Buoyant Weight}}{\text{Uplift}} = \frac{4.1}{97 \times 62.4} \times 2000 = 1.36$$

In the riverbed where the overburden load is less this factor drops to about 1.05 for pool at spillway.

3. Effect of High Uplift Pressures. - These factors of safety against flotation are very low for a dam of this magnitude and the chance of boils and springs in downstream toe area developing are quite possible

a. More serious, however, than the development of boils, etc., is the reduction of intergranular forces by the upward and opposing seepage forces. The excess pressure due to upward seepage acts to reduce intergranular stress in the same manner as the hydrostatic excess pressure from consolidation as explained in Section C, Paragraph 5 a, Page 18. Should the intergranular forces be reduced for this reason, a very serious loss of shearing strength will follow; and in view of the weak nature of the soft foundation silt and its required treatment, such a possible further reduction in shear strength is considered as requiring preventive treatment.

4. Proposed Treatment.

a. As noted above, the exact extent of this buried pervious layer is not known. Further borings would indicate its possible extent but would not guarantee the absence of an inlet connecting with the reservoir. Further borings might succeed in locating an outlet from the buried pervious deposit, if such exists; but cost of the large number of additional borings to demonstrate this conclusively is well apt to exceed cost of treatment to prevent dangerous uplift pressures in the buried pervious layer. Therefore proposed treatment has been designed on basis of the inlet and blocked outlet conditions assumed in Paragraph 2 a above.

b. Five relief wells will be installed just below downstream toe of embankment, extending through overlying impervious to tap the buried pervious layer and relieve the possible high uplift pressures therein. The buried pervious is of only small lateral extent as shown by present borings and computations indicate that 3 wells would be satisfactory. However, with such a small number of wells, the effect of one or two failing to be fully efficient would be adverse, so two additional wells were added for greater safety. With these relief wells added the factor of safety against flotation is increased to about 1.4 which is considered ample.

5. Relief Well Details.

a. Because of the depth and earth pressures involved, the use of V.C. pipe or porous concrete pipe is not considered desirable. The sections of such pipe are short and brittle and apt to dislocate under any minor horizontal foundation strains.

b. It is proposed, therefore, to use 6-inch extra strong wrought-iron pipe for the casing both for strength and life. The lower portion of the well will consist of 15 feet of brass screen of size to prevent infiltration of soils as shown on Plate No. A40.

c. The upper end of the pipe will discharge into a buried drain to prevent freezing destroying the value of wells during periods of cold weather with small or intermittent flow from the wells. The top portion of the well will be made so that any well can be cut out and tested by screwing pipe extension to the inside of 6-inch casing below the upper outlet. Location and details of the relief wells are shown on Plate No. A41.

d. To minimize any damage from possible horizontal movement in soft silt during application of dam load, the relief wells will not be installed until the third construction season.

6. Relief Well Discharge. - The quantity of seepage from each relief well may be estimated by use of Jervis' formula (10):

$$Q_{\text{per well}} = \frac{k_H H a D}{d \sqrt{E.L.}}$$

Where

k_H - average coefficient of permeability of pervious deposit in horizontal direction - 50×10^{-4} cm/sec.

H - difference in hydraulic head between pool and well outlet - 105 ft. for pool at spillway.

a - well spacing - 100 ft.

D - depth of pervious deposit - 40 ft.

d - distance between inlet of pervious deposit and wells - assumed at 5000 ft. for greater conservatism.

E.L. - extra length - 58 ft. - computed by interpolation of chart on page 44, Technical Memorandum, No. 184-1, U. S. Waterways Experiment Station.

a. In previous computation it was assumed that the pervious layer inlet was 10,000 ft. above upstream toe of dam; however, for reasons of safety in seepage load, this distance will be assumed to be 5000 ft. from relief wells which have 15 ft. screen sections and are spaced 100 ft. apart.

$$Q = \left[\frac{50}{10^4} \times \frac{1}{30.4} \right] \frac{105 \times 100 \times 40}{5000 \sqrt{58}} = 0.0136 \text{ cfs}$$

b. The value 50×10^{-4} cm/sec. may be low for k_H . Increasing by a factor of safety of 5, the discharge per well will be about

(10) W. H. Jervis (1939); "Underseepage Studies, Black Bayou Levee"; U. S. Engineer Office, Vicksburg, Mississippi.
See also "Investigation of Underseepage Lower Mississippi River Levees"; Technical Memorandum No. 184-1; U. S. Waterways Experiment Station, Vicksburg, Mississippi; October 1941.

0.08 cfs and the total for 5 wells will be about 0.40 cfs. For heads lower than full pool stage the discharge will be proportionately less.

CLAREMONT DAM
ANALYSIS OF DESIGN - APPENDIX A

INDEX OF TABLES AND PLATES

<u>TABLE NO.</u>	<u>TITLE</u>
A1	Providence District Soil Classification
A2	Triaxial Tests - Summary of Test Data - Undisturbed Varved Silts
<u>PLATE NO.</u>	
A1	General Plan
A2	Embankment Details No. 1
A3	Embankment Details No. 2
A4	Subsurface Explorations
A5	Soil Profile on Approximate ϕ of Dam
A6	Soil Section - Sta. 14+00
A7	Soil Profile through Center of Rock Valley
A8	Providence District Soil Classification
A9	Typical Samples - Taken with Shelby Tube Spoon
A10	Typical Samples - Taken with Solid Drive Spoon
A11	Typical Boring with Properties of Foundation Silt - BH-51
A12	Typical Boring with Properties of Foundation Silt - BH-10
A13	Void Ratio - Compressibility - Consolidation Test - Soft Varved Silt
A14	Consolidation - Permeability - Consolidation Test - Soft Varved Silt
A15	Consolidation - Permeability - Consolidation Test - Soft Varved Silt
A16	Void Ratio - Compressibility - Consolidation Test - Soft Varved Silt

INDEX OF PLATES (Continued)

<u>PLATE NO.</u>	<u>TITLE</u>
A17	Void Ratio - Compressibility - Consolidation Test - Soft Varved Silt
A18	Consolidation - Permeability - Consolidation Test - Soft Varved Silt
A19	Summary of Pre-consolidation Test Data
A20	Computation of Average Horizontal and Vertical Permeability of Soft Varved Silt
A21	Summary of Direct Shear Results - Soft Varved Silt
A22	Triaxial Tests - Summary of Mohr's Circles - Undisturbed Varved Silts
A23	Generalized Section
A24	Stability Analysis
A25	Contours of Hydrostatic Excess Pressure
A26	Graphical Determination of Consolidation for Non-uniform Rate of Loading
A27	Hydrostatic Excess Pressure Curves
A28	Isochrones - Consolidation - Vertical Flow
A29	Isochrones - Consolidation - Radial Flow
A30	Smeared Varved Soil Sample
A31	Effect of Combined Drainage on T_{90} for Instantaneous Loading
A32	Drain Well Backfill Gradation Limits
A33	Drain Well Theory - Formulae for Consolidation by Radial Drainage to Drain Well - Uniform Vertical Strain - Well Resistance Considered
A34	Average Stresses in Silt Due to Various Spacing of Drain Wells
A35	Drain Well Theory - Formulae for Consolidation by Radial Drainage to Drain Wells - Smear Effect Considered

INDEX TO PLATES (Continued)

<u>PLATE NO.</u>	<u>TITLE</u>
A36	Drain Well Theory - Formulae for Consolidation by Radial Drainage to Drain Well - Constant Strain - Smear and Well Resistance Considered
A37	Effect of Smear and Well Resistance on Consolidation with Drain Wells
A38	Time vs. Excess Pressure for Initial and Revised Loading Rates
A39	Formula for Seepage Under Dike with Natural Blanket over Pervious Layer
A40	Range of Grain Sizes of Material in Deeply Buried Pervious Deposit
A41	Embankment Drainage System - Plan and Details

PROVIDENCE DISTRICT SOIL CLASSIFICATION

CLASS	DESCRIPTION OF MATERIAL
1	<u>Graded from Gravel to Coarse Sand.</u> — Contains little medium sand.
2	<u>Coarse to Medium Sand.</u> — Contains little gravel and fine sand.
3	<u>Graded from Gravel to Medium Sand.</u> — Contains little fine sand.
4	<u>Medium to Fine Sand.</u> — Contains little coarse sand and coarse silt.
5	<u>Graded from Gravel to Fine Sand.</u> — Contains little coarse silt.
6	<u>Fine Sand to Coarse Silt.</u> — Contains little medium sand and medium silt.
7	<u>Graded from Gravel to Coarse Silt.</u> — Contains little medium silt.
8	<u>Coarse to Medium Silt.</u> — Contains little fine sand and fine silt.
9	<u>Graded from Gravel to Medium Silt.</u> — Contains little fine silt.
10	<u>Medium to Fine Silt.</u> — Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.
10C	<u>Medium Silt to Coarse Clay.</u> — Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.
11	<u>Graded from Gravel or Coarse Sand to Fine Silt.</u> — Contains little coarse clay.
12	<u>Fine Silt to Clay.</u> — Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt.
12C	<u>Clay.</u> — Contains little silt. Possesses behavior characteristics of clay.
13	<u>Graded from Coarse Sand to Clay.</u> — Contains little fine clay (colloids). Possesses behavior characteristics of silt.
13C	<u>Clay.</u> — Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay.

CL A. OF D.

TABLE NO. A2

WAR DEPARTMENT															CLAREMONT DAM CORPS OF ENGINEERS, U. S. ARMY															
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	
Sample and Test Number	Type of Test (S) Constant Principal Stress	Size of Specimen Diameter cm Height cm Slenderness Ratio H ₀ /D ₀	Initial Condition of Specimen (M) Medium (U) Undisturbed (R) Remolded (C) Consolidated (P) Plastic Limit not determined	Water Content W ₀ %	Degree of Saturation G ₀ %	Void Ratio e ₀	Void Ratio at Preliminary Consolidation e _c	Void Ratio at End of Test e ₀	Saturation Test Preliminary Consolidation e _c	Time allowed for Compression from 90% to 95% of max. (G ₀ -G ₁) Δt min	Strain at End of Test max. (G ₀ -G ₁) %	Strain at End of Test max. (G ₀ -G ₁) %	Void Strain at End of Test max. (G ₀ -G ₁) %	Water Content at End of Test max. (G ₀ -G ₁) %	Ultimate Stress Ratio max. (G ₀ -G ₁) %	Measured Slip of Shear Plane %	Sketch showing Type of Failure	Piston-cap contact Rigid (R) Flexible (F)	Remarks											
																				Diagrams (a) Quick control (b) Quick release (c) Quick release (d) Quick release (e) Quick release (f) Quick release (g) Quick release (h) Quick release (i) Quick release (j) Quick release (k) Quick release (l) Quick release (m) Quick release (n) Quick release (o) Quick release (p) Quick release (q) Quick release (r) Quick release (s) Quick release (t) Quick release (u) Quick release (v) Quick release (w) Quick release (x) Quick release (y) Quick release (z)										
BH-54 UC 3	(S)	σ ₃	15	3.48	6.66	1.92	(M) (U)		28.0	108.8	0.721	0.669	0.633	703	3020	13,085	16,905	10	14.3	6.4	25.1	54.4	4.45					(F)		
UC 3	(S)	σ ₃	45																											
UC 3	(S)	σ ₃	100																											
UC 7	(S)	σ ₃	15	3.65	5.84	1.60	(M) (U)		27.3	111.2	0.683	0.605	0.644	274	23,044	43,043	50,631	4.8	14.0	9.8	22.2	56.7	4.3					(F)	Drainage accidentally closed during partial test	
UC 7	(S)	σ ₃	45																											
UC 7	(S)	σ ₃	100																											
UC 9	(S)	σ ₃	15																											
UC 9	(S)	σ ₃	45																											
UC 9	(S)	σ ₃	100	3.30	9.62	2.91	(M) (U)		27.9	114.0	0.675	0.658	0.707	1185	10,080	33,625	44,355	11.8	15.5	10.8	33.9	43.2	3.5					(F)		
UC 12	(S)	σ ₃	15	3.30	10.26	3.11	(M) (U)		35.03	101.0	0.954	0.875	0.853	2700	2880	12,720	12,720	15.5	15.5	10.8	33.9	43.2	3.5					(F)		
UC 12	(S)	σ ₃	45	3.30	8.96	2.72	(M) (U)		20.9	100.0	0.565	0.610	0.526	998	1,440	18,530	18,632	6.6	6.8	28.8	18.7	16.0	4.3					(F)		
UC 12	(S)	σ ₃	100	3.25	9.64	2.98	(M) (U)		21.9	106	0.562	0.570	0.582	2756	14,475	61,850	63,060	12.0	12.1	33.2	17.8	39.5	4.7					(F)	Sample compacted after test. Final test not run	

Triaxial Tests
Summary of Test Data
Undisturbed Varved Silts

SL No. CL3-F9

ENGINEERING DIVISION - SOILS LABORATORY

CL A. of D.

PROVIDENCE, R. I.
TABLE NO. A2

Triaxial Tests
Summary of Test Data
Undisturbed Varved Silts

SL No. CLS-F9

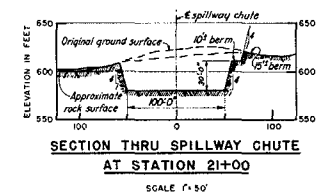
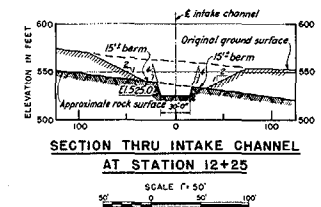
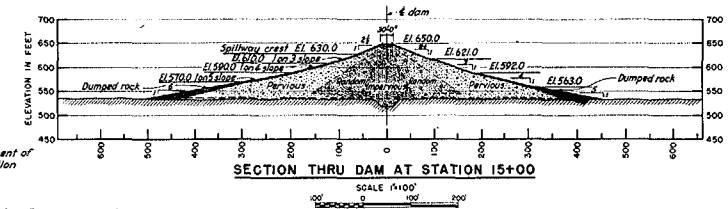
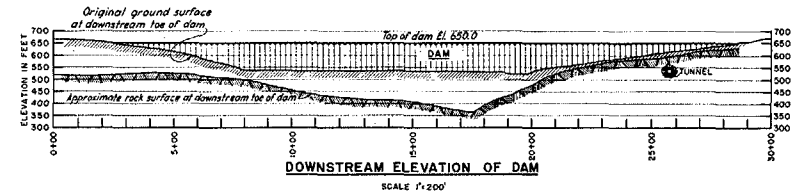
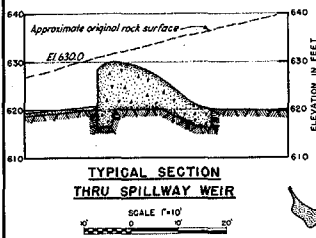
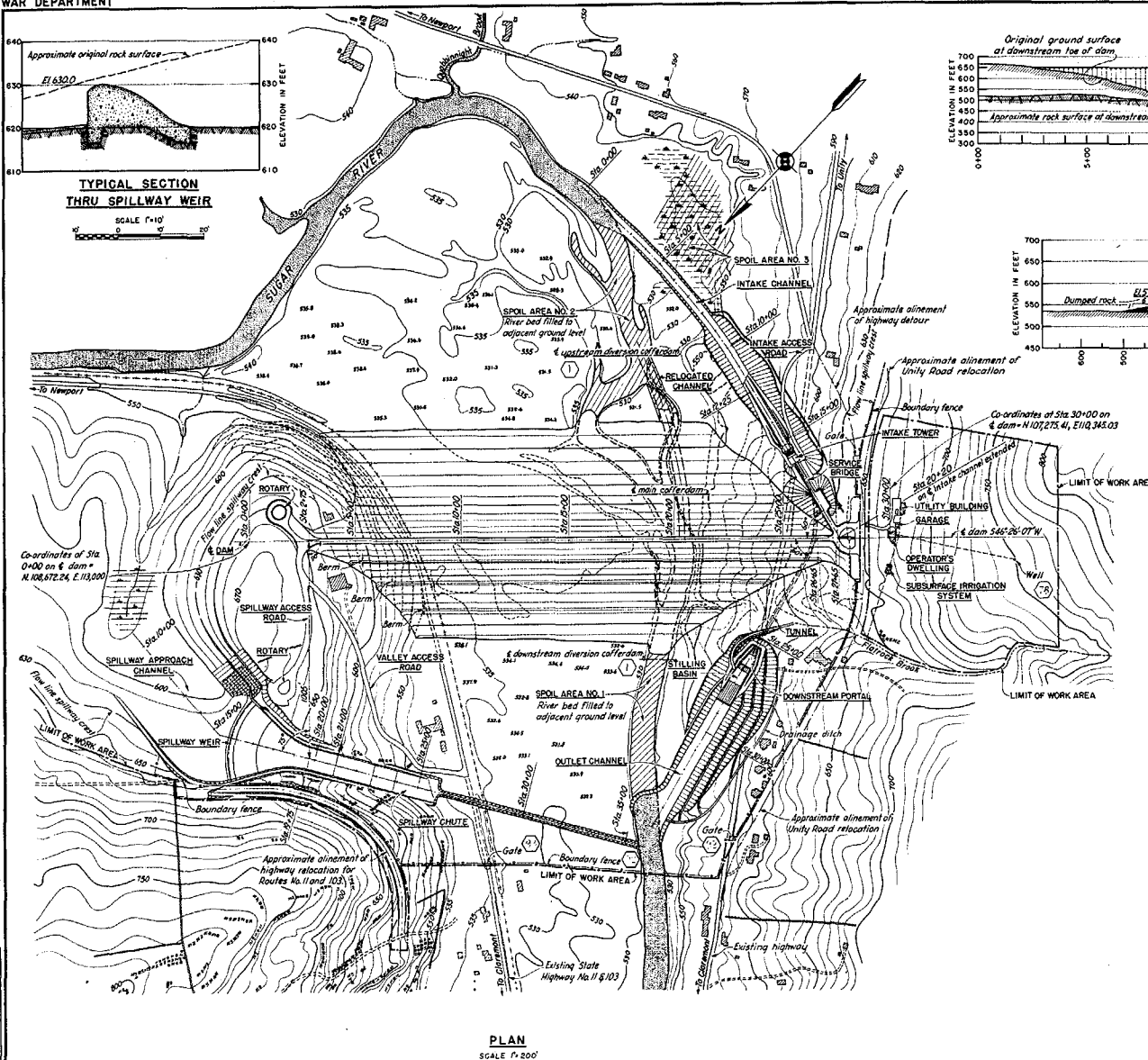
PROVIDENCE, R. I.
TABLE NO. A2

ENGINEERING DIVISION - SOILS LABORATORY

CL A. OF D.

CL A. OF D.

PLATE NO. A1

**NOTES**

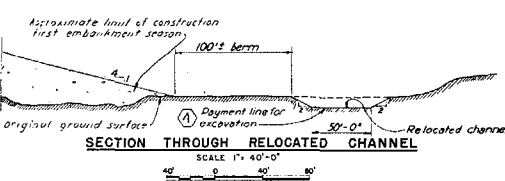
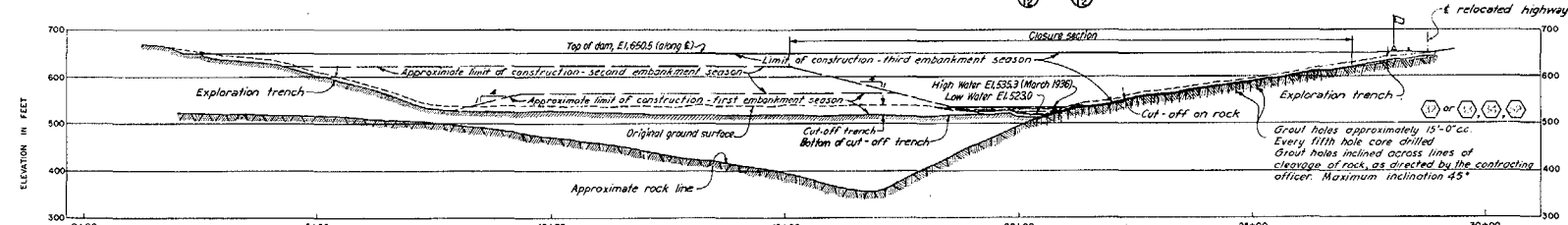
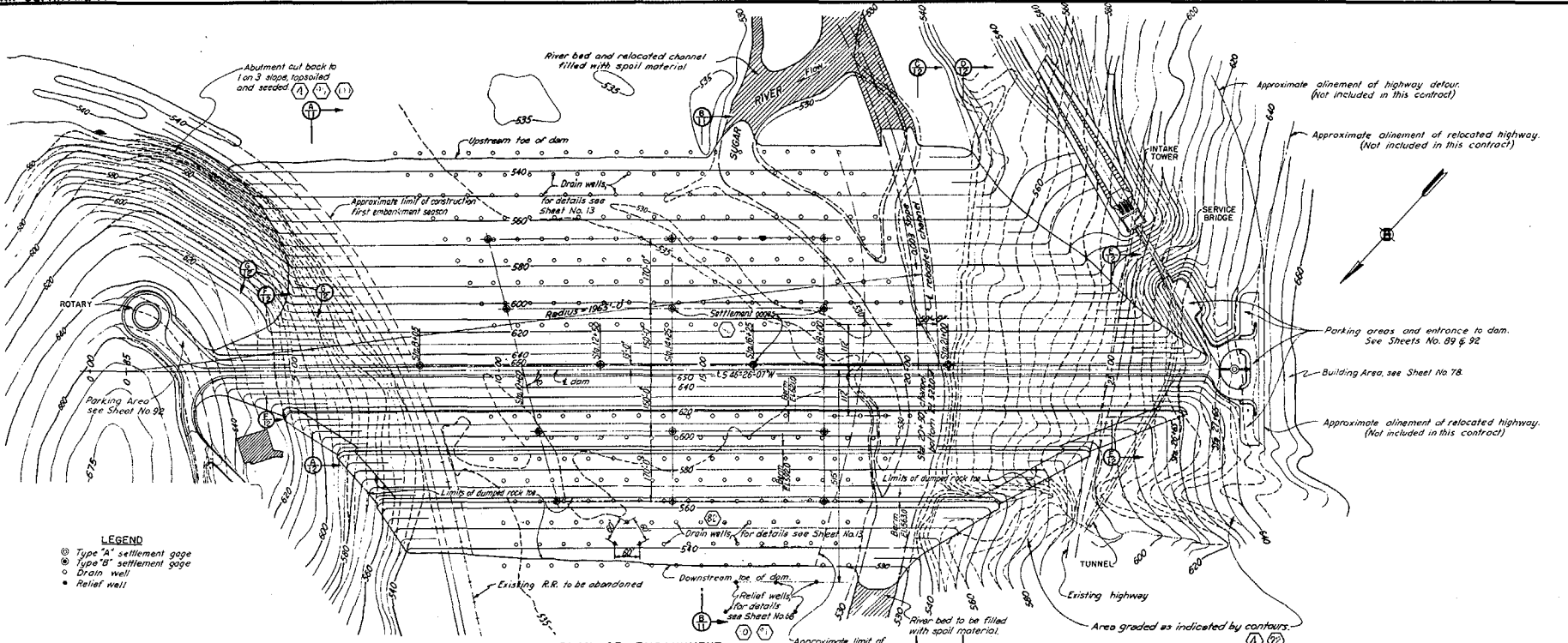
- Elevations refer to Mean Sea Level Datum.
Contour interval is ten feet except as shown.
Spill area.
For location of existing buildings to be removed see Sheet No. 4.

CONNECTICUT RIVER FLOOD CONTROL			
CLAREMONT DAM			
GENERAL PLAN			
SUGAR RIVER		NEW HAMPSHIRE	
IN 94 SHEETS	SCALE: 1 IN=200 FT.	SHEET NO. 2	
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945			
SUBMITTED	APPROVAL RECOMMENDED	APPROVED	
C. D. VAN VORST	C. D. VAN VORST	C. D. VAN VORST	
PROJECT ENGINEER	SENIOR ENGINEER	CHIEF ENGINEER	
PREPARED	DRAWN	CHECKED	
A. W. NICHOLS	A. W. NICHOLS	A. W. NICHOLS	
PROJECT	TRACED	FILE NO. GT-1-1779	

KEY	DATE	REVISION (Indicated by Δ)	REVIEWED BY	APPROVED BY

CL A.O.F.D.

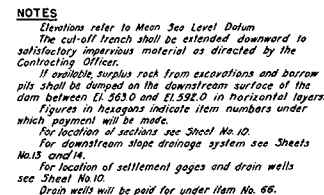
PLATE NO. 2



NOTES
 For downstream slope drainage system, see Sheets No. 13 and 14.
 Elevations refer to Mean Sea Level Datum.
 Contour interval five feet.
 The cut-off trench, shall be excavated to satisfactory impervious material as directed by the contracting officer.
 The longitudinal limits of the cut-off trench, exploration trench, and cut-off on rock will be determined in the field.
 Figures in hexagons indicate item numbers under which payment will be made.
 For details of settlement gages see Sheet No. 68.
 The number and location of drain wells is subject to change.

KEY	DATE	REVISION (indicated by Δ)	REVIEWED BY	APPROVED BY

CONNECTICUT RIVER FLOOD CONTROL	
CLAREMONT DAM	
EMBANKMENT DETAILS NO. 1	
SUGAR RIVER	NEW HAMPSHIRE
IN 94 SHEETS	SHEET NO. 10
SCALE: 1" = 100 FT.	
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN 1945	
SUBMITTED	APPROVAL RECOMMENDED
PROJECT ENGINEER	CHIEF ENGINEER
PREPARED	DRAWN
PROJECT UNIT NO. 1	FILE NO. CT-1-1780



KEY DATE REVISION (Indicated by Δ) REVISION CLBY AP BY

CONNECTICUT RIVER FLOOD CONTROL
CLAREMONT DAM
EMBANKMENT DETAILS NO. 2

SUGAR RIVER NEW HAMPSHIRE
IN 94 SHEETS SCALE: 1 IN. = 40 FT. SHEET NO. 11

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTALS
DESIGNED BY *W. H. H. H.* APPROVED BY *W. H. H. H.*
DRAWN BY *W. H. H. H.* CHECKED BY *W. H. H. H.*
PROJECT ENGINEER *W. H. H. H.* COL. COMD'G U.S. ENGINEER
DISTRICT OFFICE

PREPARED BY *W. H. H. H.* DRAWN: G. V. V.
TRACED: B. G. V.
CHECKED: *W. H. H. H.*

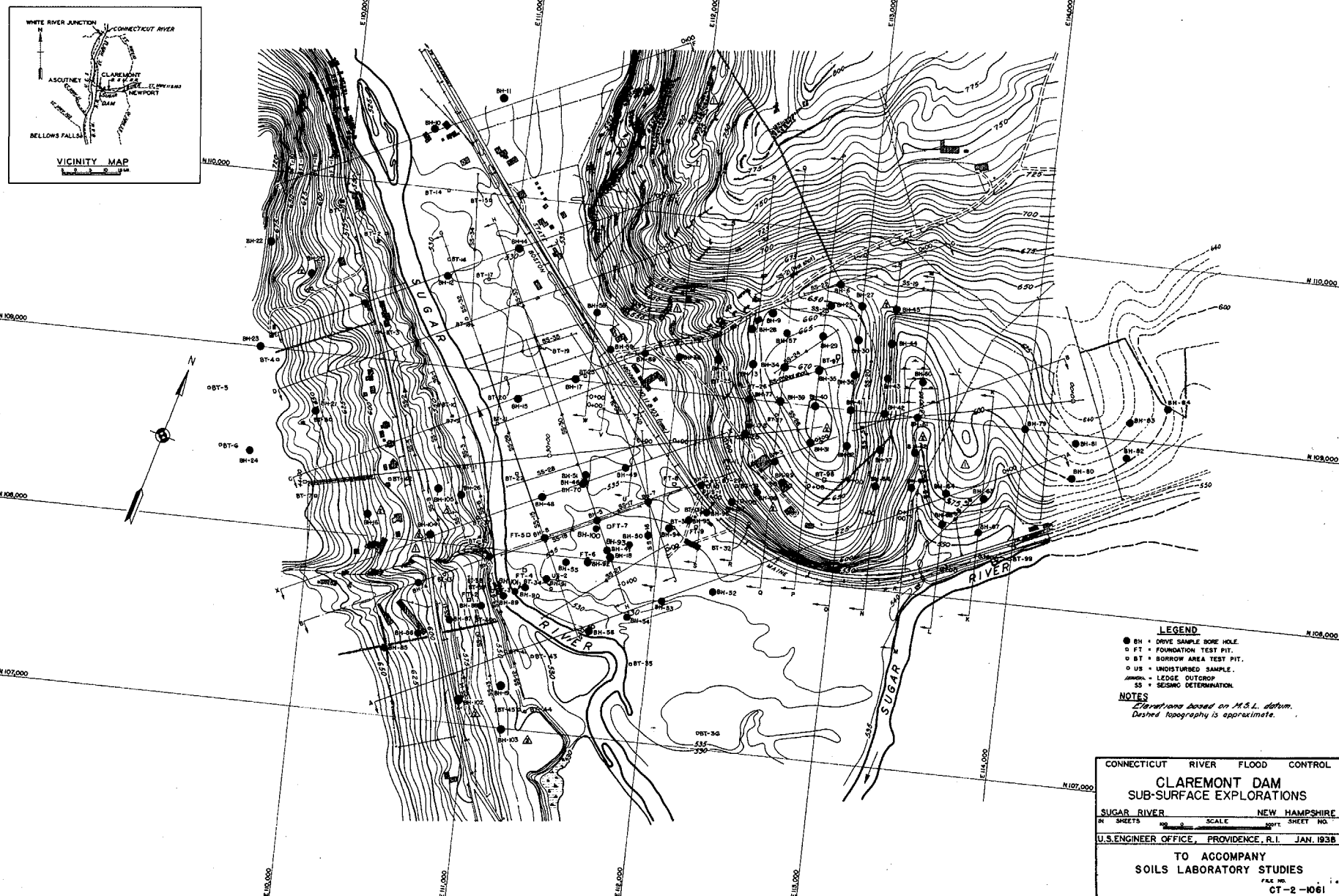
FILE NO. CT-1-1781

CL A OF D.

PLATE NO. A4

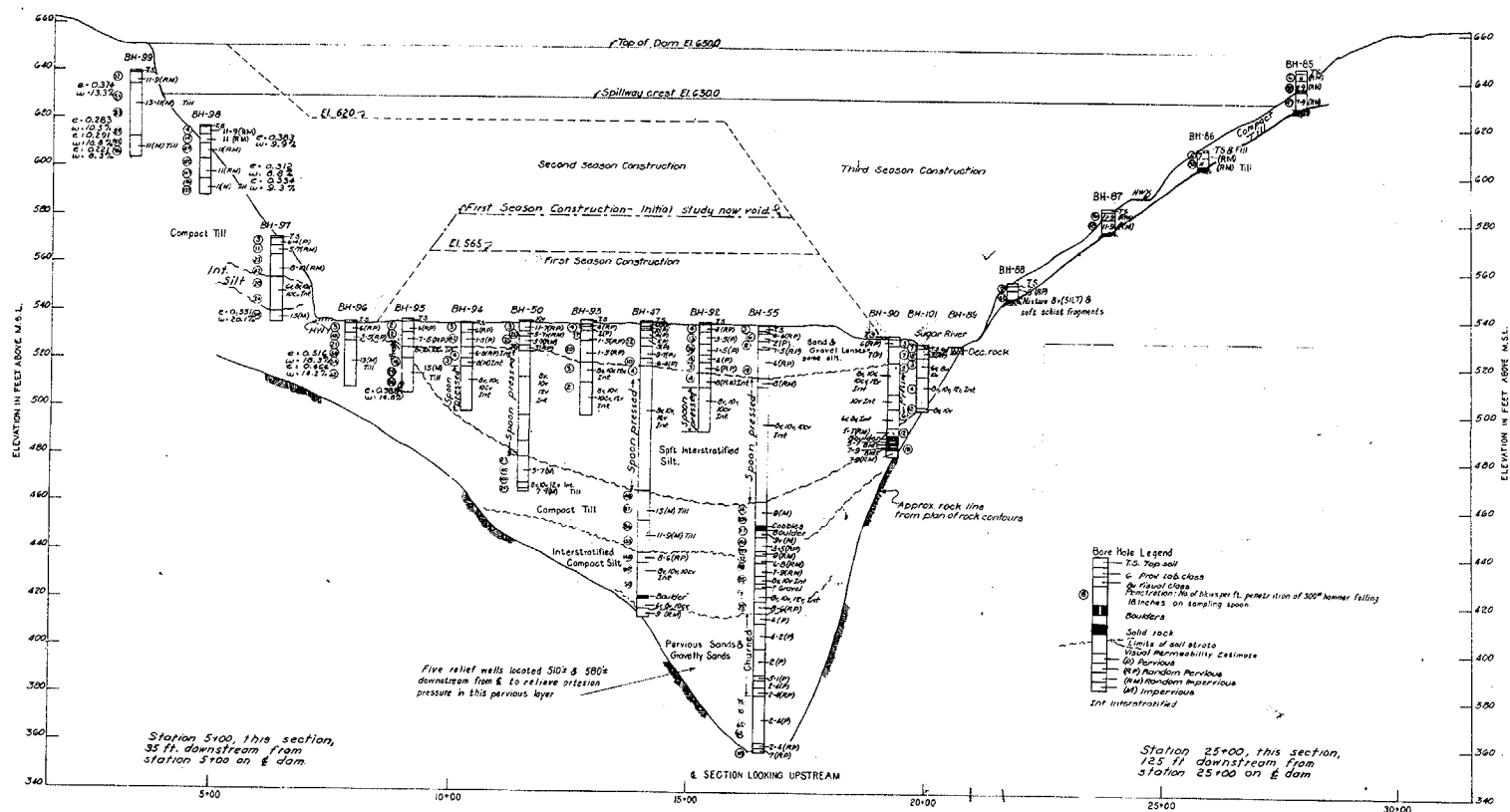
WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



CL A.O.F.D.

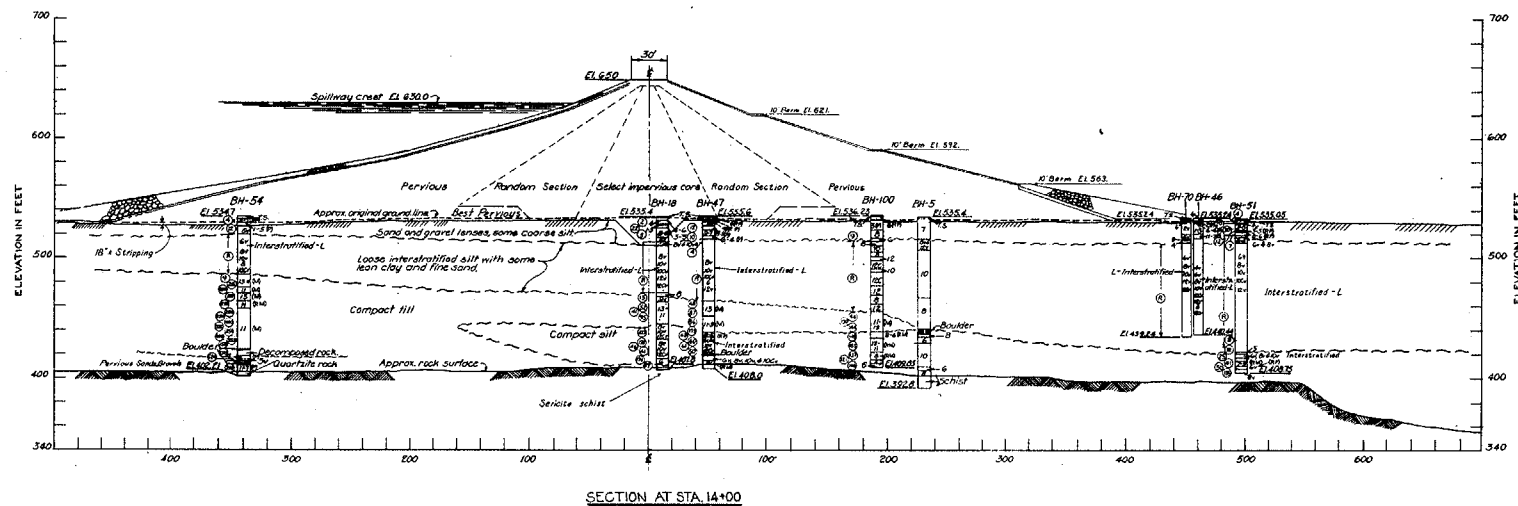
PLATE NO. A5



Note: Section not parallel to center line of dam. Horizontal stations not to indicated scale.

KEY	DATE	REVISION (Indicated by Δ)	REVIEW (OK BY) AD BY

CONNECTICUT RIVER FLOOD CONTROL	
CLAREMONT DAM	
SOIL PROFILE ON APPROXIMATE E. OF DAM	
SUGAR RIVER	NEW HAVEN SHEET
IN SHEETS	SCALE: 1" = 100' VERT 1" = 20'
SHEET NO.	
U.S. ENGINEER OFFICE, PROVIDENCE, R.I.	
SUBMITTED BY	SOILS LABORATORY STUDY
DESIGNED BY	ENGINEER
CHECKED BY	ENGINEER
APPROVED BY	ENGINEER
TRACED BY	ENGINEER
FILE NO.	SL NO. CLS - A29



Core Hole Legend

1. Type

2. Lab. class & visual permeability

3. Visual class

4. Penetration: No. of blows per ft. penetration of 300" hammer falling 10 inches on sampling spoon.

5. Boulder

6. Limits of soil strata

7. Penetration by Russian method, or pressed by hand.

8. Layers

9. Visual Permeability Estimate

10. Pervious

11. Random Pervious

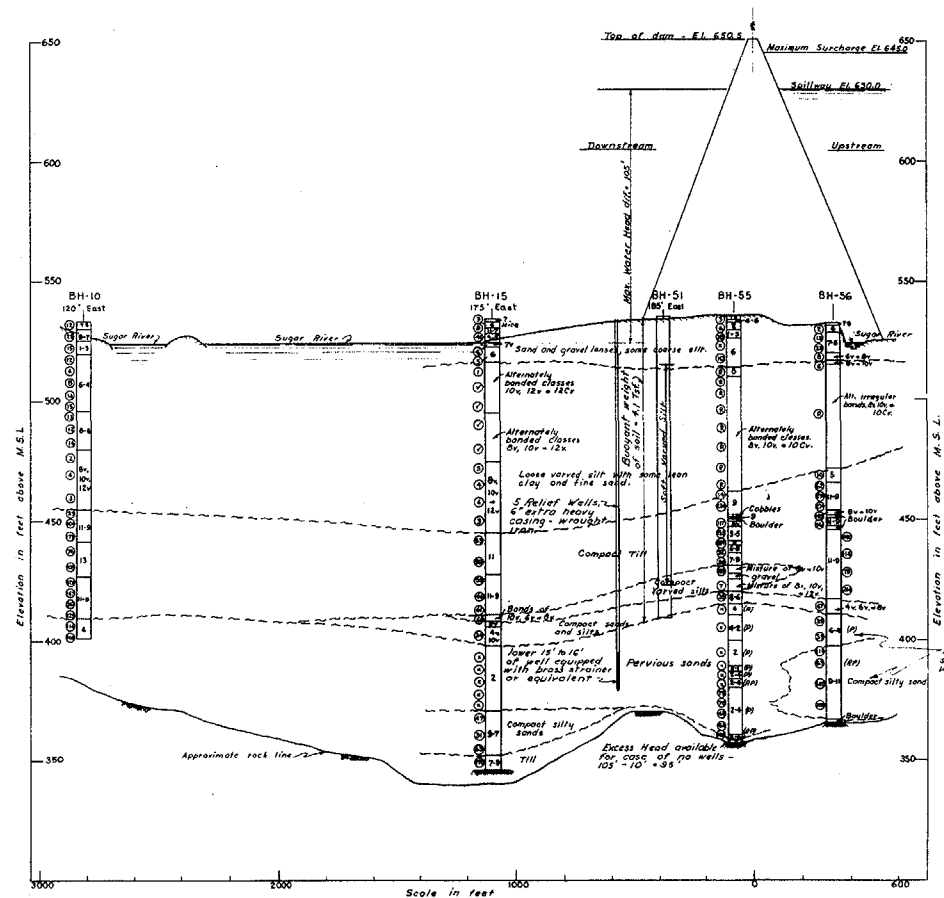
12. Random Impervious

13. Impervious

Top of rock

KEY	DATE	REVISION (indicated by Δ)	REVIEW	BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL	
CLAREMONT DAM	
SOILS SECTION - STA. 14+00	
SUGAR RIVER	NEW HAMPSHIRE
IN SHEETS	SCALE: HORIZ. 1"=40' VERT. 1"=40' SHEET NO.
U.S. ENGINEER OFFICE, PROVIDENCE, R.I.	
SUBMITTED	SOILS LABORATORY STUDY
FORWARDED	FILE NO. CLS-A27D
PREPARED	FILE NO.
CHECKED	



LEGEND

- 13 Topsoil
6a Persistence Classification
6b Visual Classification
- ⊙ Penetration: No. of blows per ft. penetration of 300 # hammer, falling 18" on sampling spoon.
 - ⊙ 3.5' under seed of hammer and rods.
 - ⊙ By churning.
 - ⊙ Russian Method - pressed down
- Limits of soil strata.
- [P] - Pervious
[RP] - Random Pervious

CLAREMONT DAM
SOIL PROFILE
THROUGH CENTER OF ROCK VALLEY

IN 1 SHEET SCALE HOR. 1" = 200' SHEET NO. 1

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.,

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.,	
SUBMITTED: <i>H. S. Lane</i>	SOILS LABORATORY STUDY

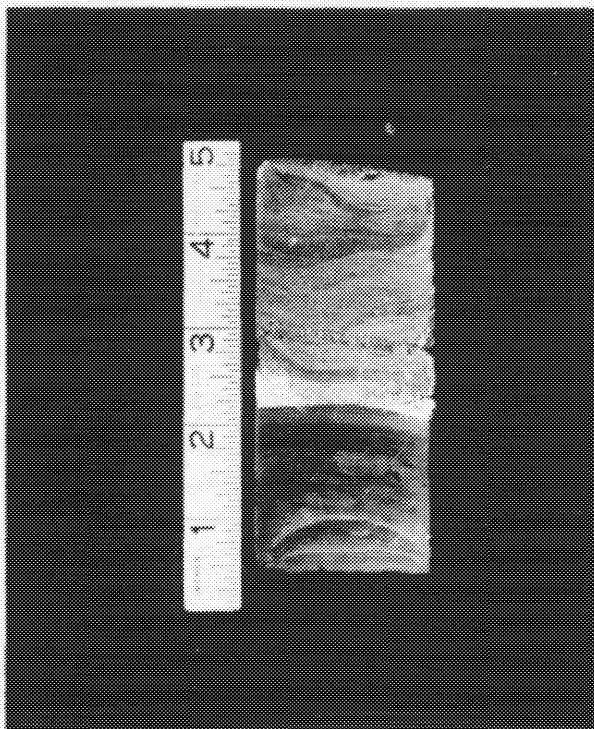
ENGINEER		
HEAD, SOILS LABORATORY		
PREPARED	DRAWN: R. D. L.	S.L. NO. GLS-A28
<i>R. D. L.</i>	TRACED:	FILE NO.
SOILS LABORATORY	CHECKED: <i>R. D. L.</i>	

[illegible]

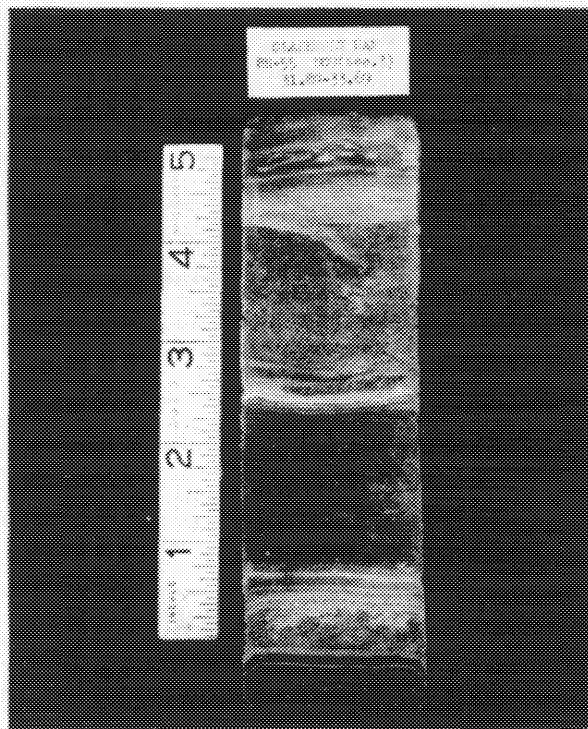
The graph shows the relationship between grain size and percentage finer by weight. The x-axis represents grain size in millimeters (logarithmic scale from 20.0 to 0.0001) and the number of mesh per inch (Tyler Standard). The y-axis represents the percentage finer by weight (0 to 100). The graph includes curves for various soil classes (1-13) and a dashed line for 'Gravel and larger'. The graph is used to determine soil classification based on grain size distribution.

Gravel	Coarse Sand	Medium Sand	Fine Sand	Coarse Silt	Medium Silt	Fine Silt or Coarse Clay	Medium Clay	Fine Clay (Colloids)
	Class 2		Class 6		Class 10 or 10 C			
	Class 4		Class 8		Class 12 or 12 C			
	Class 1	Class 3	Class 5	Class 7	Class 9	Class 11	Class 13 or 13 C	

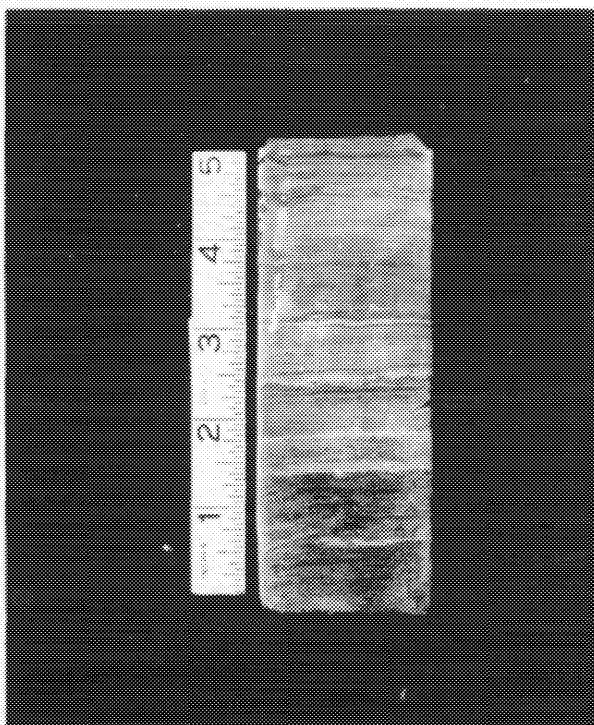
DIAGRAM SHOWING LIMITS OF SOIL CLASSES



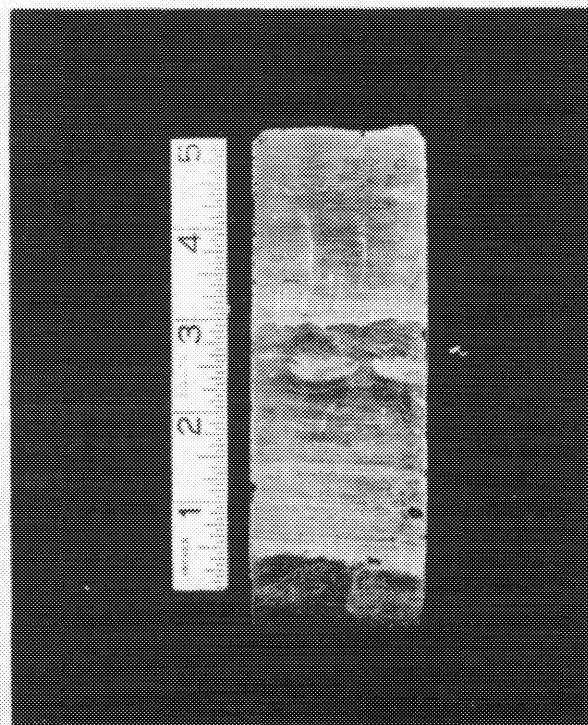
CLAREMONT DAM BH-55,UC9(See.2)SLD 955
Irregularly banded coarse to fine silt
and lean clay.



CLAREMONT DAM BH-55,UC9(See.3)SLD 956
Alternate bands and lenses of fine
sand to fine silt and lean clay.

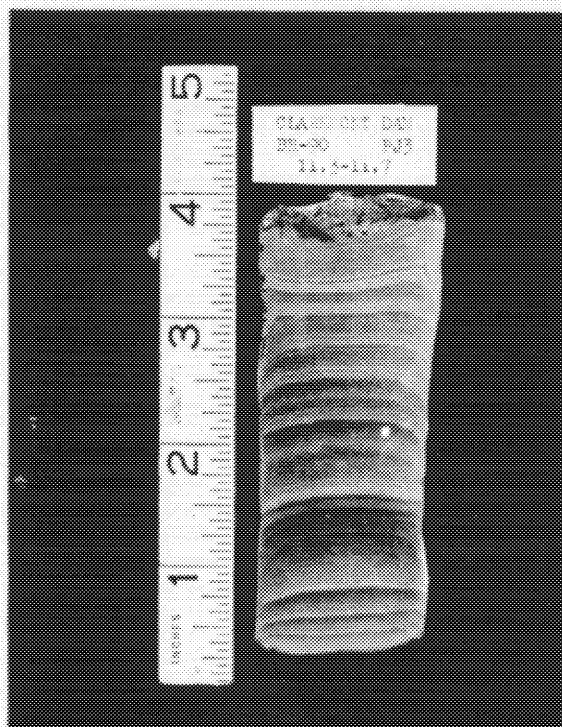


CLAREMONT DAM BH-55,UC19(See.2)SLD 962
Indistinct bands of medium to fine
silt.

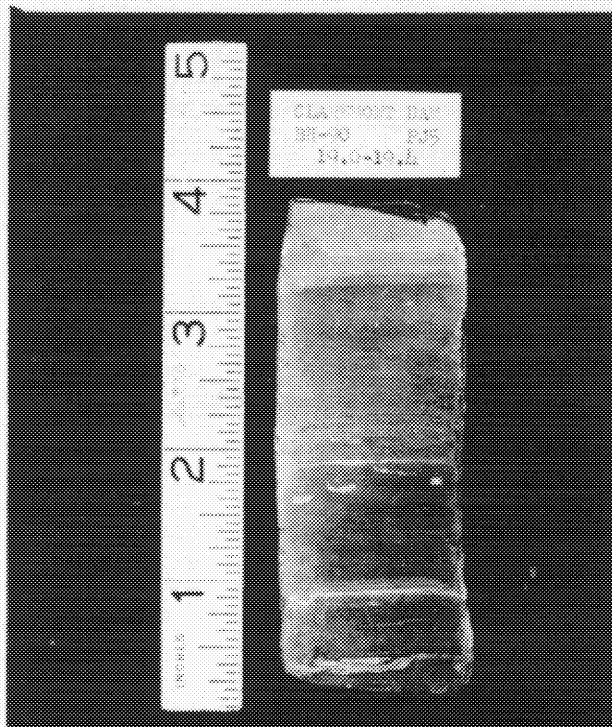


CLAREMONT DAM BH-55,UC19(See.3) SLD 963
Indistinct bands of medium to fine silt.

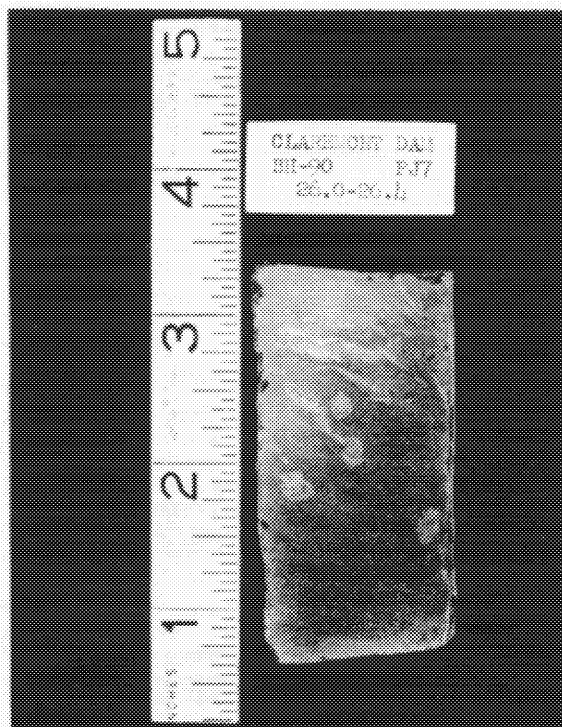
TYPICAL SAMPLES - TAKEN WITH SHELBY TUBE SPOON



CLAREMONT DAM BH-90, PJ3 SLD 927
Alternately banded medium to fine
silt.



CLAREMONT DAM BH-90, PJ5 SLD 929
Alternately banded medium silt to
lean clay.



CLAREMONT DAM BH-90, PJ7 SLD 930
Medium to fine silt.



CLAREMONT DAM BH-90, PJ9 SLD 931
Irregular banded medium sand to medium
silt.

TYPICAL SAMPLES - TAKEN WITH SOLID DRIVE SPOON

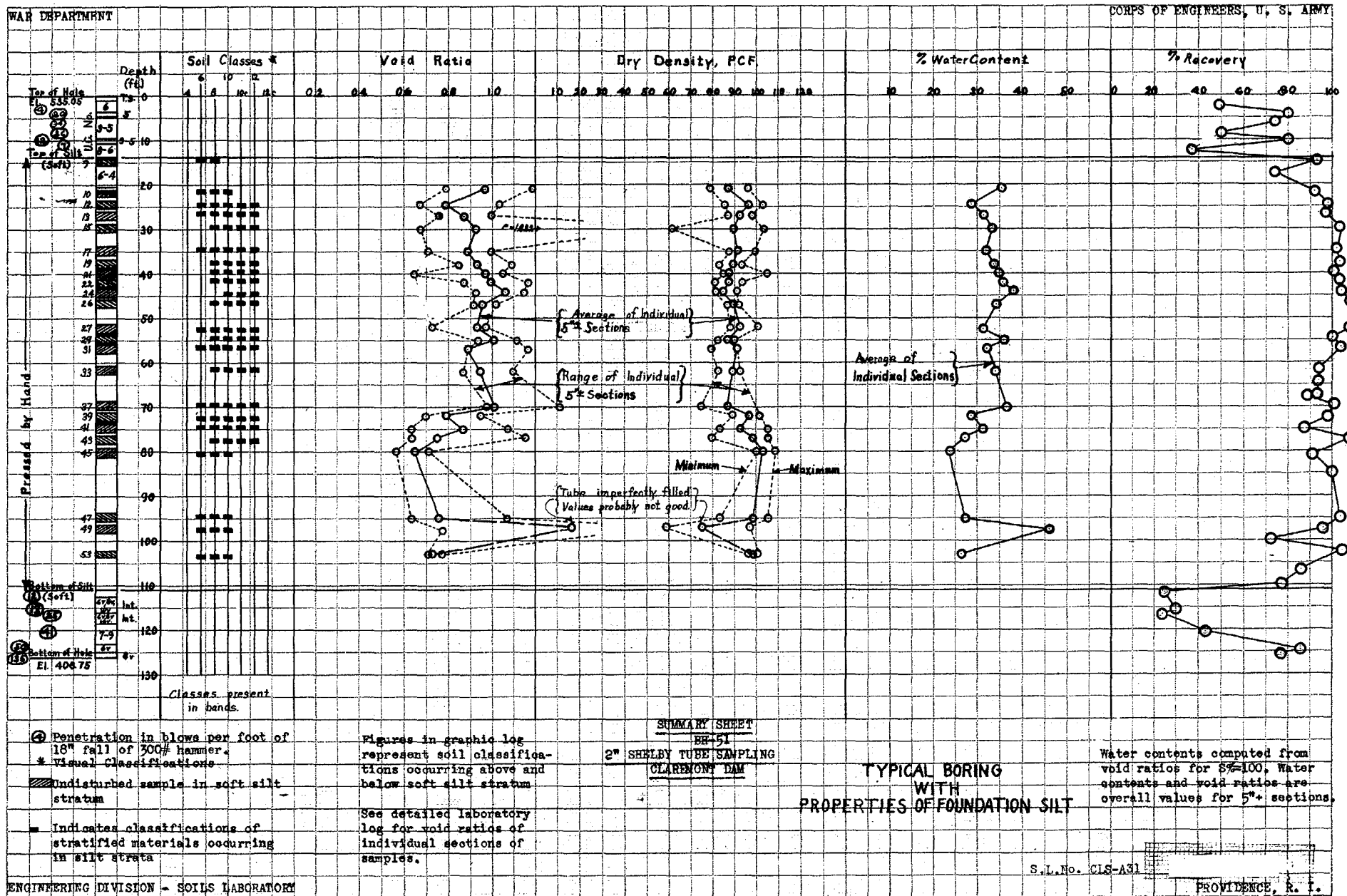
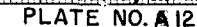
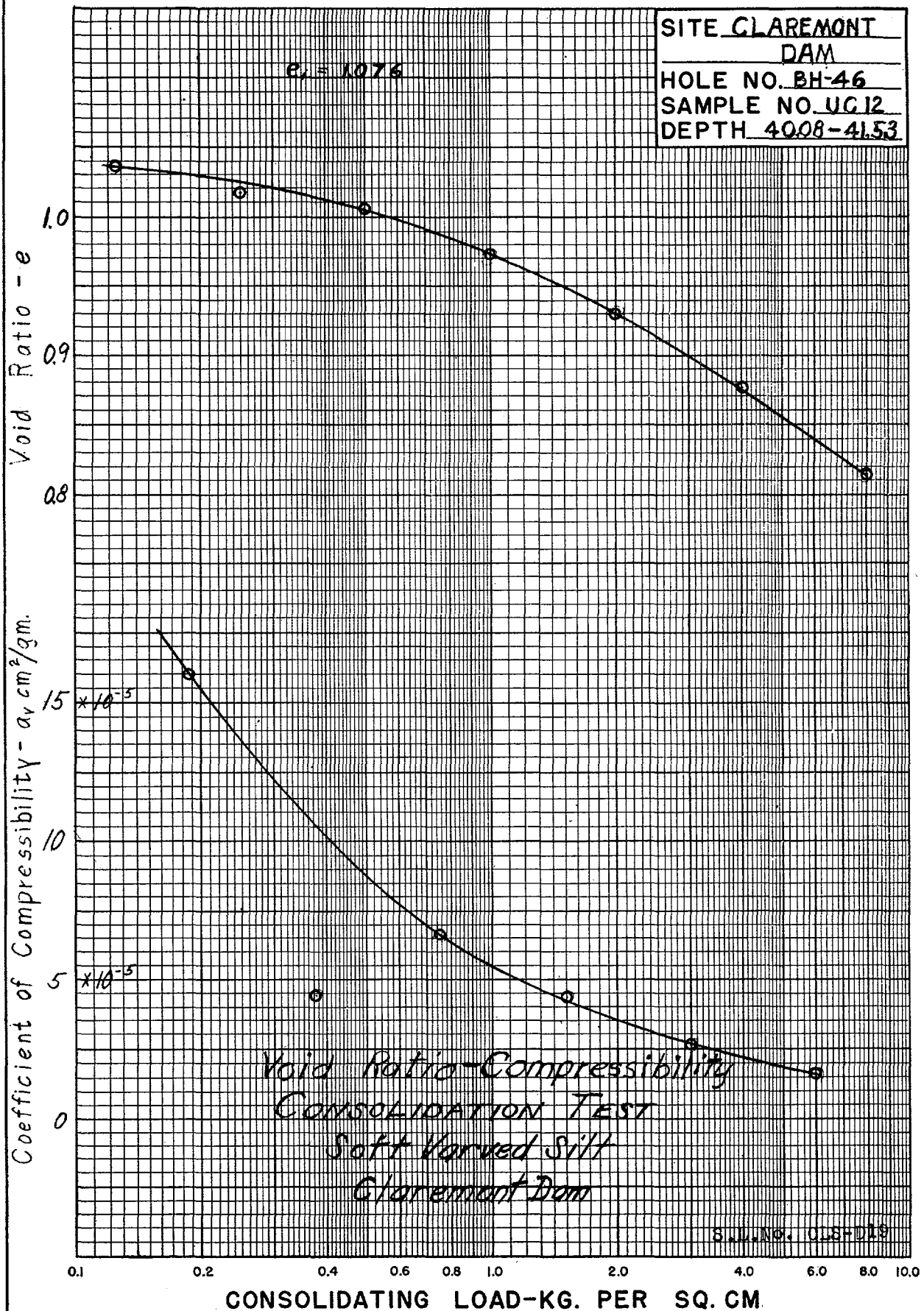


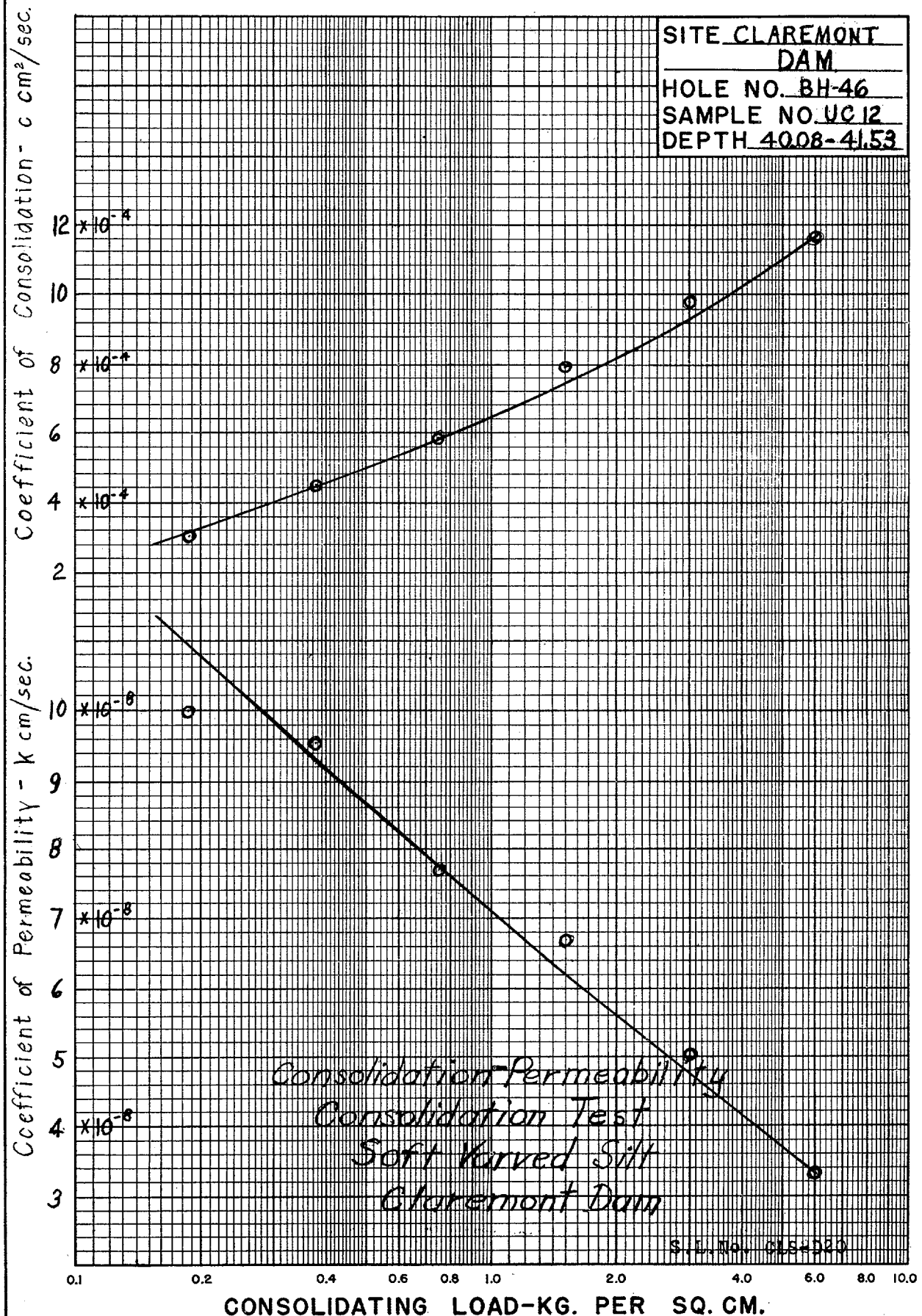
PLATE NO. A12



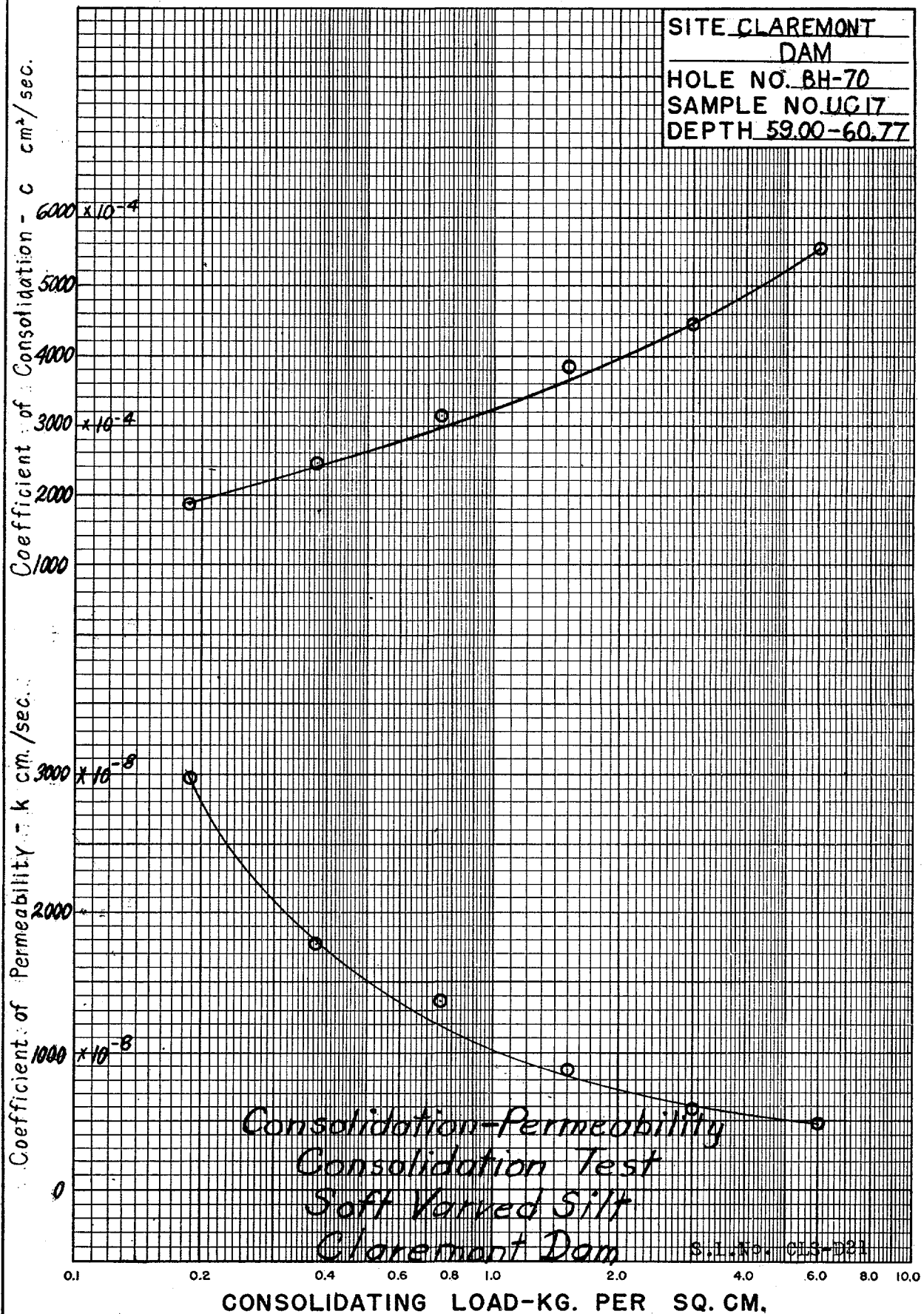
CONSOLIDATION CHARACTERISTICS



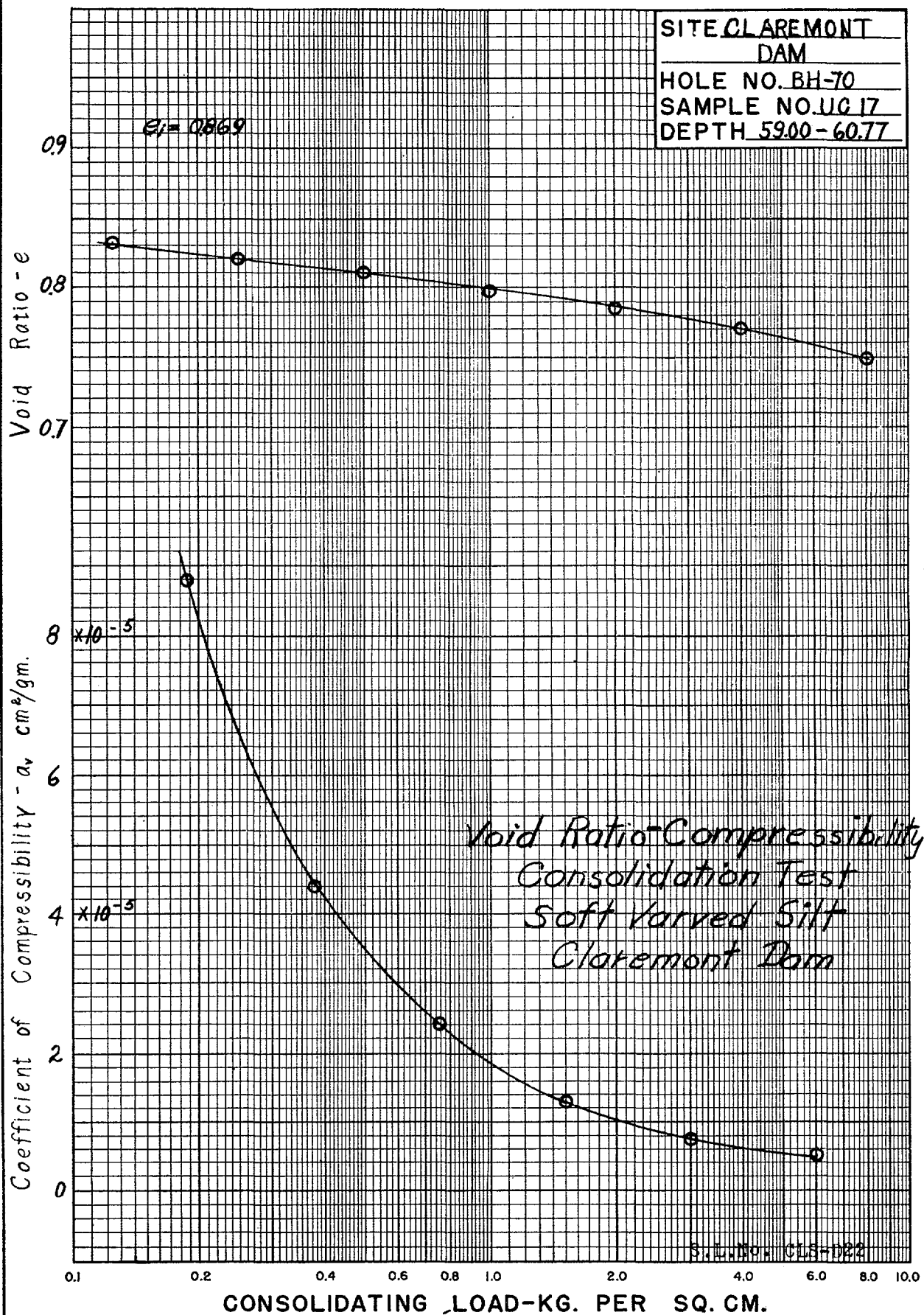
CONSOLIDATION CHARACTERISTICS



CONSOLIDATION CHARACTERISTICS



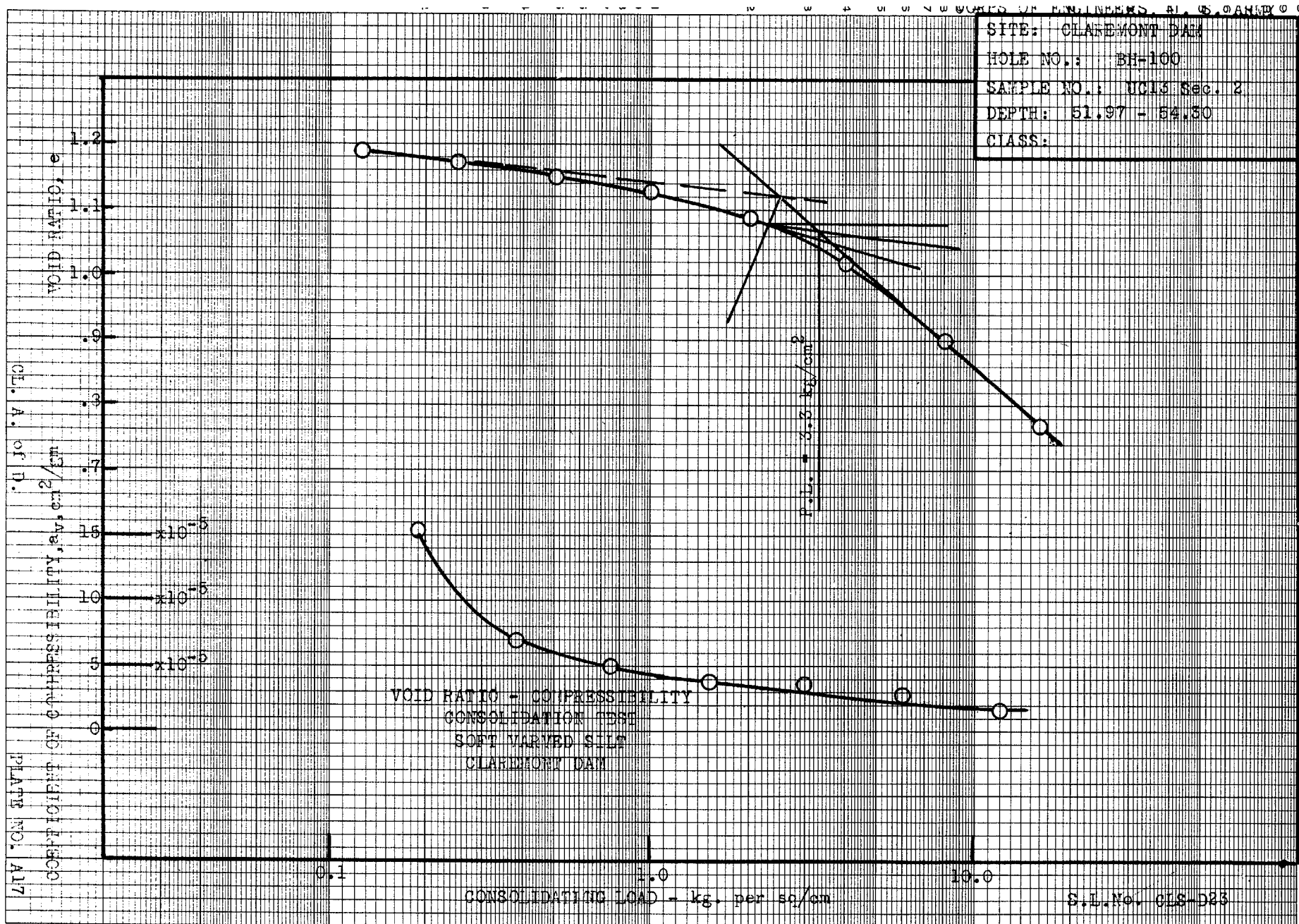
CONSOLIDATION CHARACTERISTICS



CL

A. OF D.

PLATE NO. A17

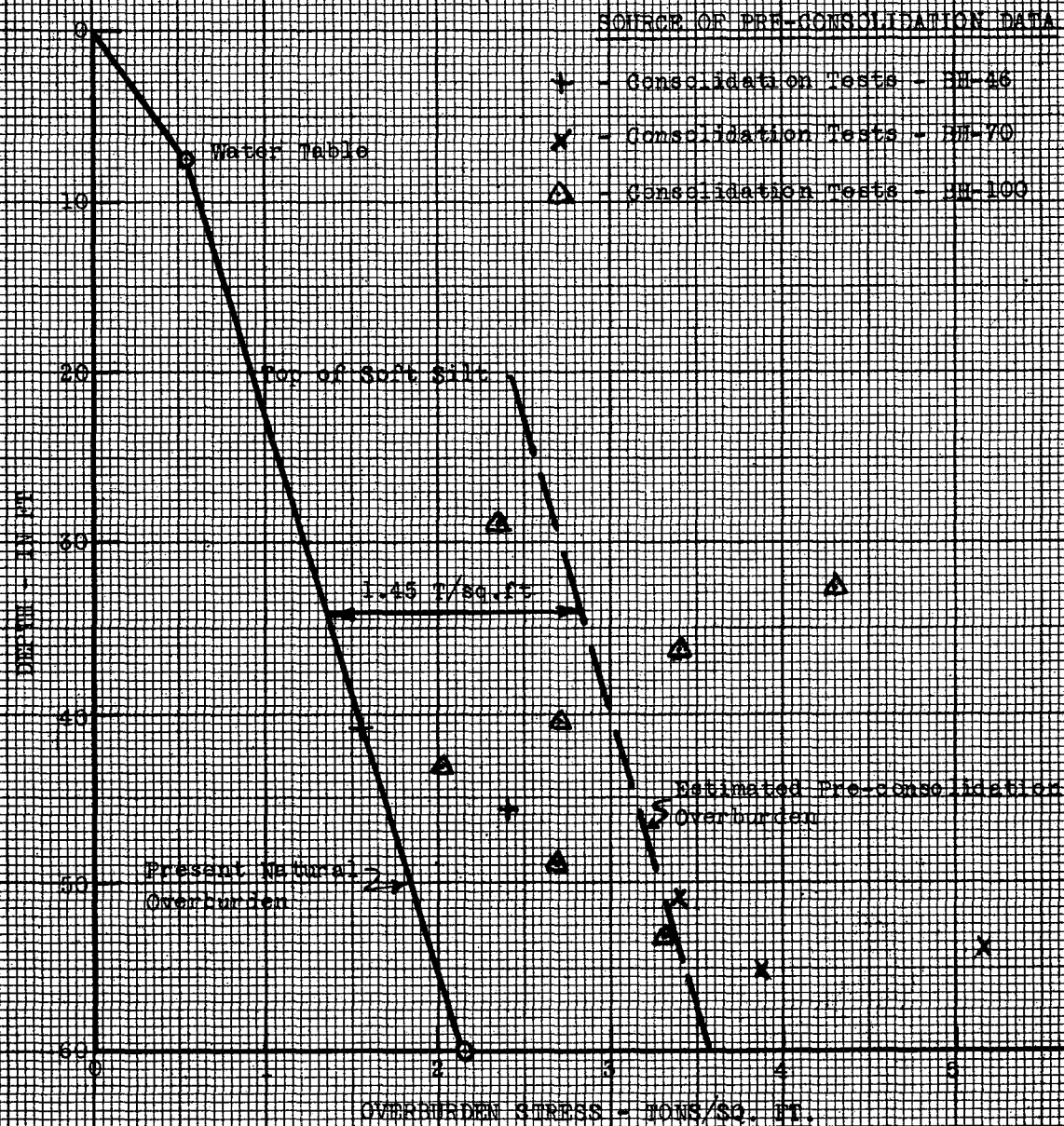


CLASS:



S.L.No. C-8-024

PLATE NO. A18

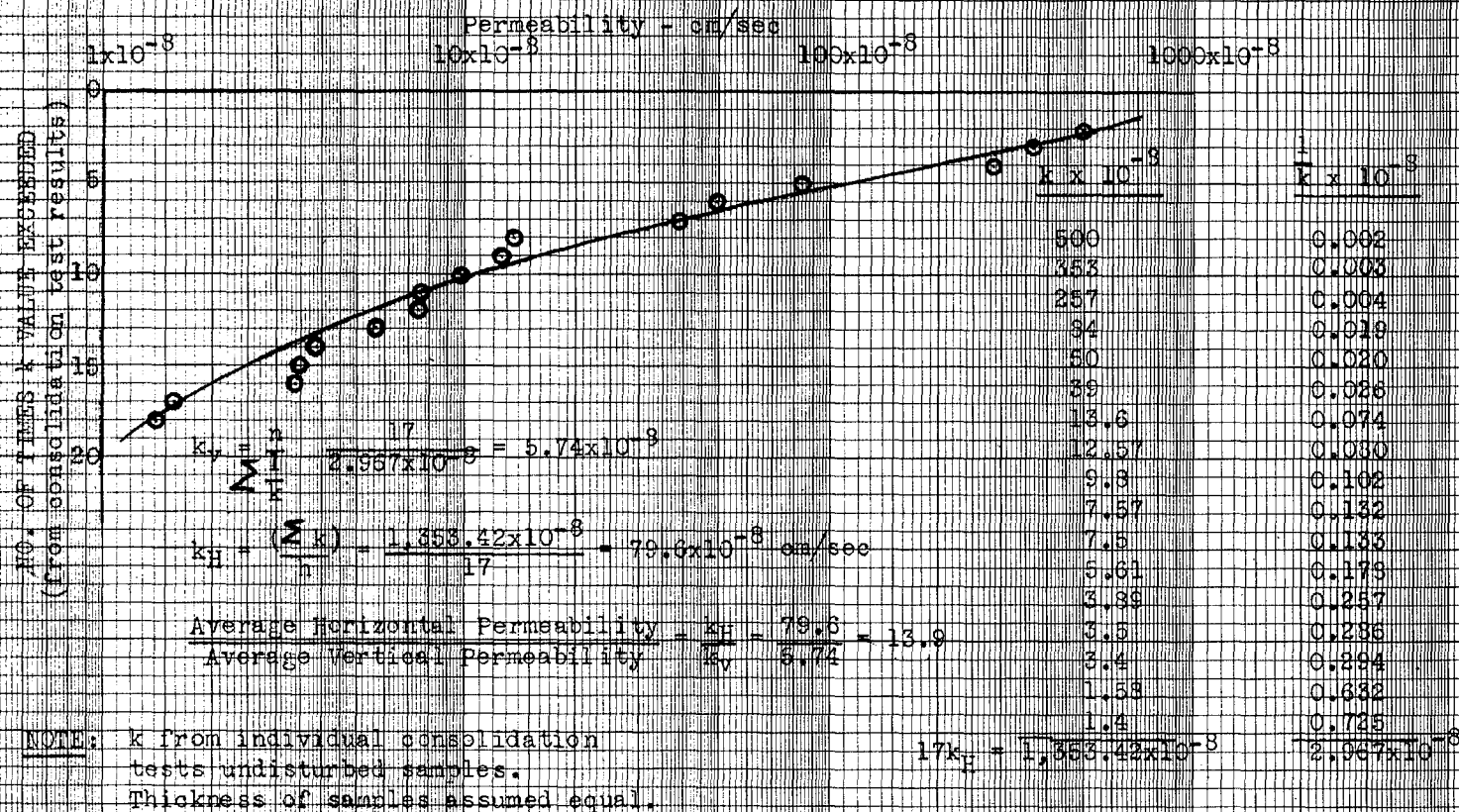


CLAREMONT DAM
SUMMARY OF PRE-CONSOLIDATION
TEST DATA

S.L. No. CLS-217

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



COMPUTATION OF AVERAGE
HORIZONTAL AND VERTICAL PERMEABILITY
OF SOFT VARVED SILT

S.L.No. CLS-D16

ENGINEERING DIVISION - SOILS LABORATORY

PROVIDENCE, R. I.

SHEAR TEST

SUPPLEMENTARY DATA

Rate of strain _____
 Consolidation FULL
 Shear Plane SATURATED
 Remarks SLOW SHEAR. STARTED
TEST AT 2½ TURNS PER INCREMENT.
WAITING FOR 100% CONSOLIDATION
THEN APPLYING ANOTHER INCREMENT
OF LOAD & DECREASING TURNS AS
SAMPLE APPROX-CLASS
CHED ULTIMATE LOAD.
 $\phi =$ _____ $c =$ _____ tons per sq. ft.

SITE CLAREMONT DAM

HOLE NO. AS SHOWN

SAMPLE NO. AS SHOWN

DEPTH _____

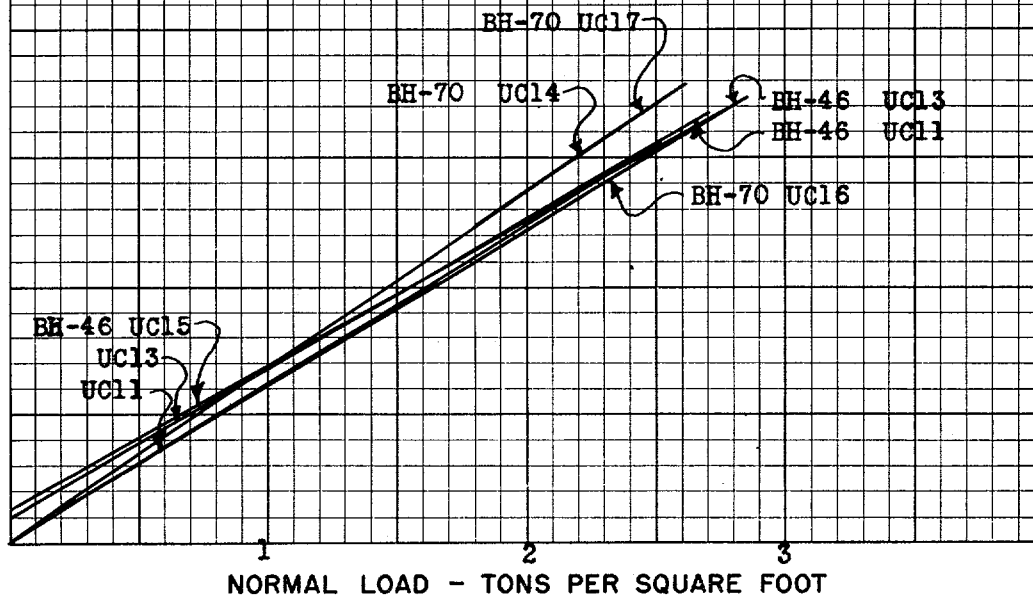
TYPICAL DIRECT SHEAR CURVES IN
SOFT SILT

SUMMARY OF DIRECT SHEAR
RESULTS - SOFT VARVED SILT

ULTIMATE SHEARING STRENGTH - TONS PER SQUARE FOOT

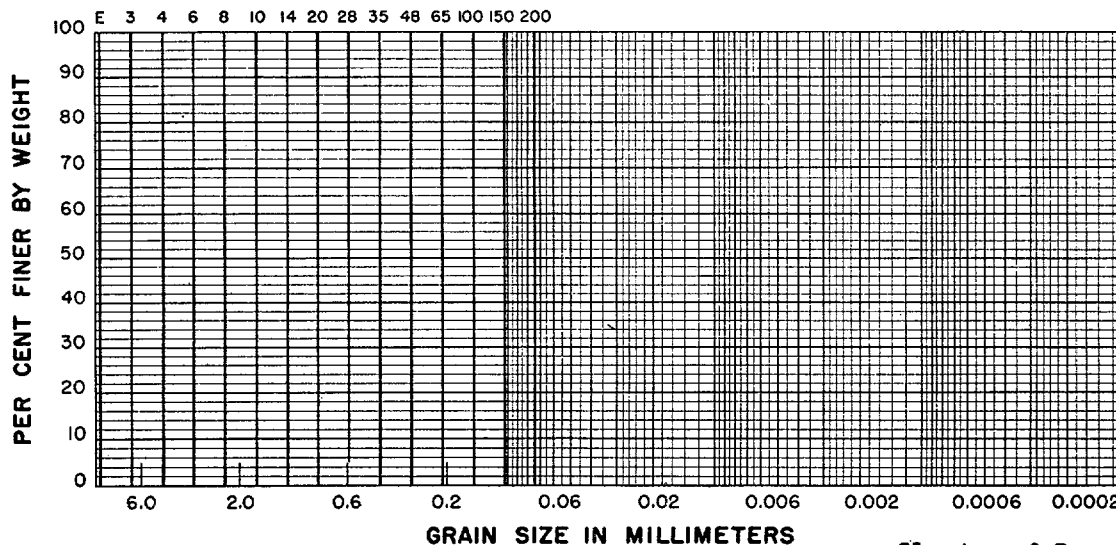
Range of ϕ 27° - 30° - 36° - 0°

Range of c 0.0-0.12



NO. MESH PER INCH

S.L.No. CLS-G4



U. S. GOVERNMENT PRINTING OFFICE 147703

CL. A. of D.

S.L. FORM NO. 66

CL A. OF D.

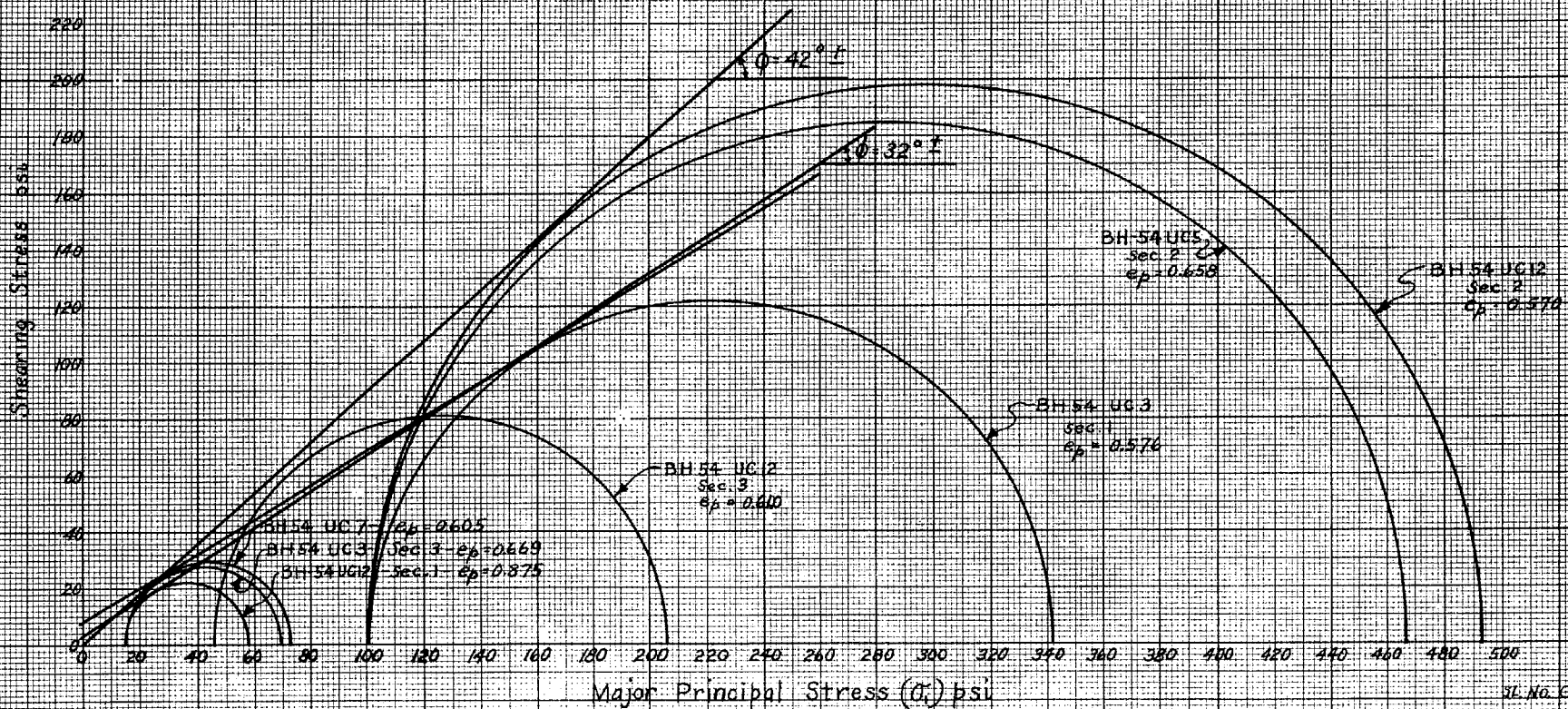
PLATE NO. A21

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

Site: Claremont Dam
Hole No. BH-54

Triaxial Test Data

Method: Constant σ_3 , slow
consolidatedNote: See remarks and data
on Table No. A2Triaxial Tests
Summary of Mohr Circles
Undisturbed Varved SiltsENGINEERING DIVISION - SOILS LABORATORY
PRINTED IN U.S.A.

CL. A. OF D.

CL No. C15-F10
PROVIDENCE, R. I.
PLATE NO. A22
EUGENE DIEZGEN CO. NO. 346 DX

CL A.O.F.D.

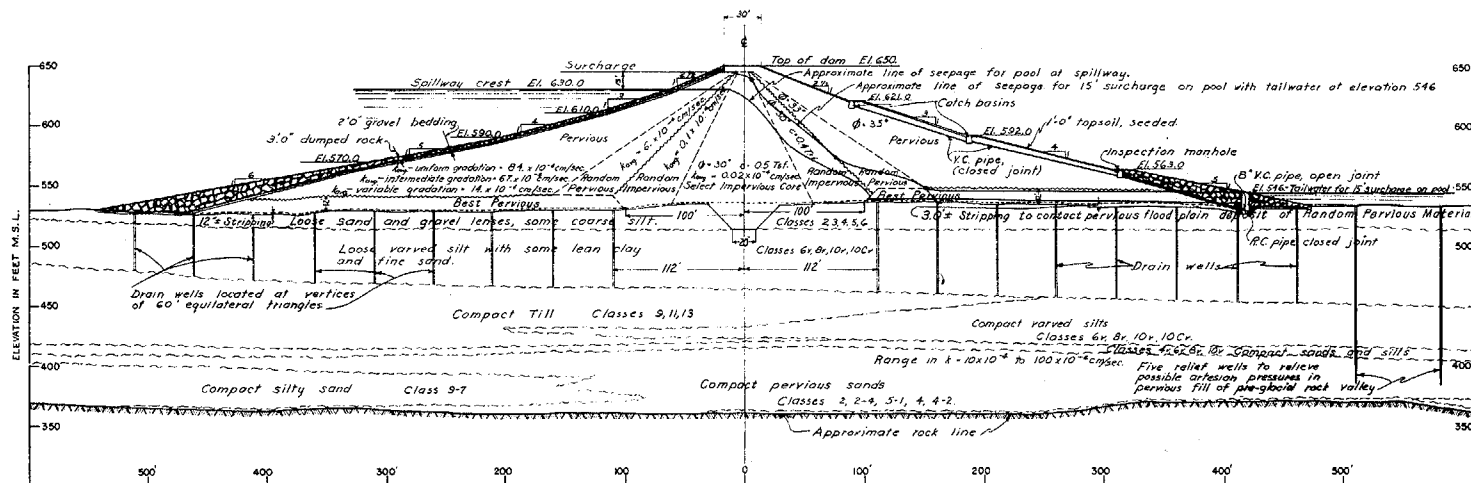
CLAREMONT DAM
GENERALIZED SECTION

PLATE NO. A23

KEY	DATE	REVISION (Indicated by Δ)	REV'D BY	CHK'D BY	APP'D BY

CONNECTICUT RIVER FLOOD CONTROL	
CLAREMONT DAM GENERALIZED SECTION	
SUGAR RIVER	NEW HAMPSHIRE
IN 1 SHEETS	SCALE 1"=40' SHEET NO. 1
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN 1945	
DESIGNED BY <i>R. H. Lane</i>	SOILS LABORATORY STUDY
HEAD, SOILS LABORATORY	PREPARED BY <i>R. H. Lane</i>
DRAWN: R.D.L.	SL NO. CLS - A 30
TRACED: R.D.L.	FILE NO.
CHECKED: R.D.L.	

CL A OF D.

PLATE NO. A24

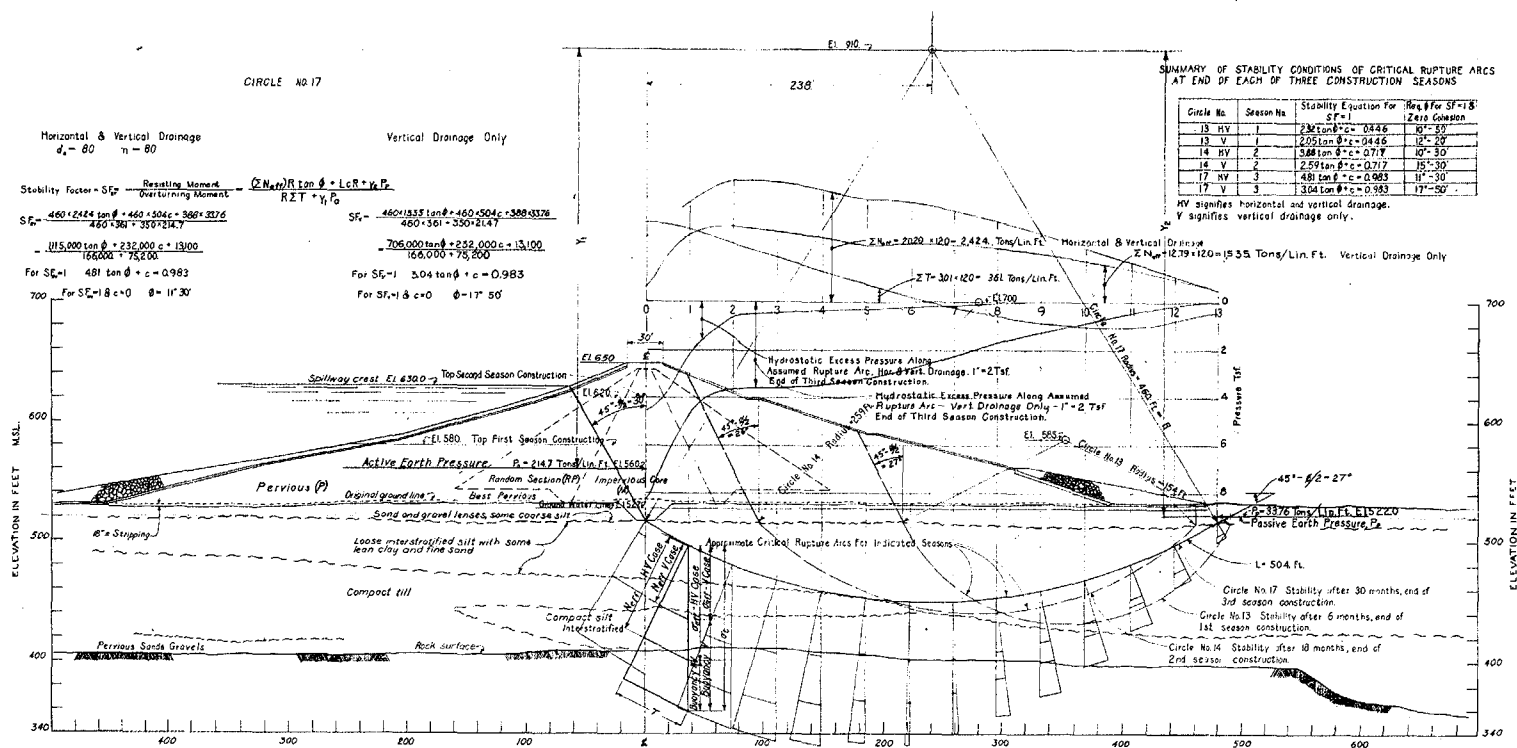
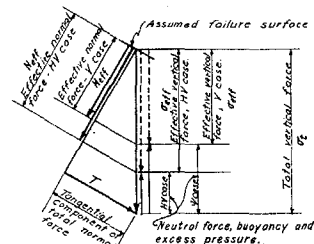


TABLE OF PHYSICAL PROPERTIES

MATERIAL	Specific Gravity	Moist. Wt. (%)	Dry Wt. (%)	Unit Weights - PCF	Angle of Friction	Angle of Cohesion		
Impervious Concrete	2.76	136	112	128	142	144	81.5	30°
Random Section (includes floodplain dep.)	2.73	261	92	135	147	148	85.5	33°
Pervious Sand Gravel	2.72	305	67	130	138	145	82	36°
Foundation Silt	2.75	905	-	88	-	121	58	-
Rip Rap	-	-	-	130	-	-	-	-

Position	Moist. Wt. (%)		Moist. Wt. (%)		Solvent-PCF		Silt		Rising		Depth		Density		Total Load		Density		Density	
	192.50		144.50		130.50		145.50		121.50		10.00		10.00		10.00		10.00		10.00	
	Height	Weight	Height	Weight	Height	Weight	Height	Weight	Height	Weight	Height	Weight	Height	Weight	Height	Weight	Height	Weight	Height	Weight
0	119	845	11	79	0	5	34	0	0	0	0	11	34	4.5	9.58	4.84	B.24	4.65		
1	7	1334	0	47	345	22	132	11	60	21	127	0	32	100	2.15	10.35	3.15	4.71	5.71	
2	0	0	0	67	141	40	276	11	0	37	224	0	48	150	7.65	10.21	2.15	3.65	5.15	
3	0	0	0	31	228	58	400	11	80	49	296	0	60	189	0.4	10.04	2.28	3.60	5.48	
4	0	0	0	6	44	71	490	11	80	58	351	0	69	216	0.35	9.65	2.51	3.42	5.58	
5	0	0	0	0	64	442	11	80	64	338	0	0	75	234	0.30	9.08	2.64	3.20	5.54	
6	0	0	0	0	59	407	11	80	67	405	0	0	78	244	0.21	8.92	2.65	2.78	5.22	
7	0	0	0	0	49	335	11	80	67	405	0	0	78	244	0.10	8.22	2.54	2.25	4.69	
8	0	0	0	0	40	276	11	80	63	351	0	0	74	231	0.03	7.36	2.34	1.69	4.00	
9	0	0	0	0	27	166	11	80	57	345	7	46	65	212	0	6.56	2.12	1.25	3.37	
10	0	0	0	0	15	111	11	80	48	290	9	55	50	154	0	5.36	1.54	0.67	2.70	
11	0	0	0	0	7	48	11	80	36	215	11	72	47	147	0	4.16	1.47	0.48	1.95	
12	0	0	0	0	5	34	11	80	19	115	6	39	30	94	0	2.65	0.94	0.15	1.09	
13	0	0	0	0	7	48	11	80	0	0	0	0	11	34	0	1.28	0.34	0	0.34	

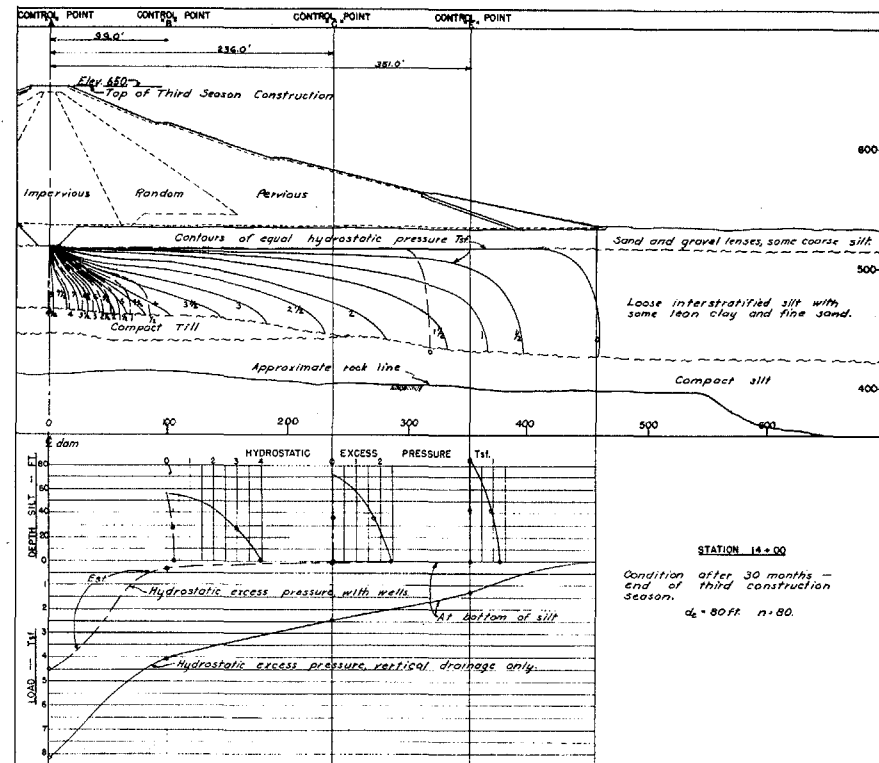
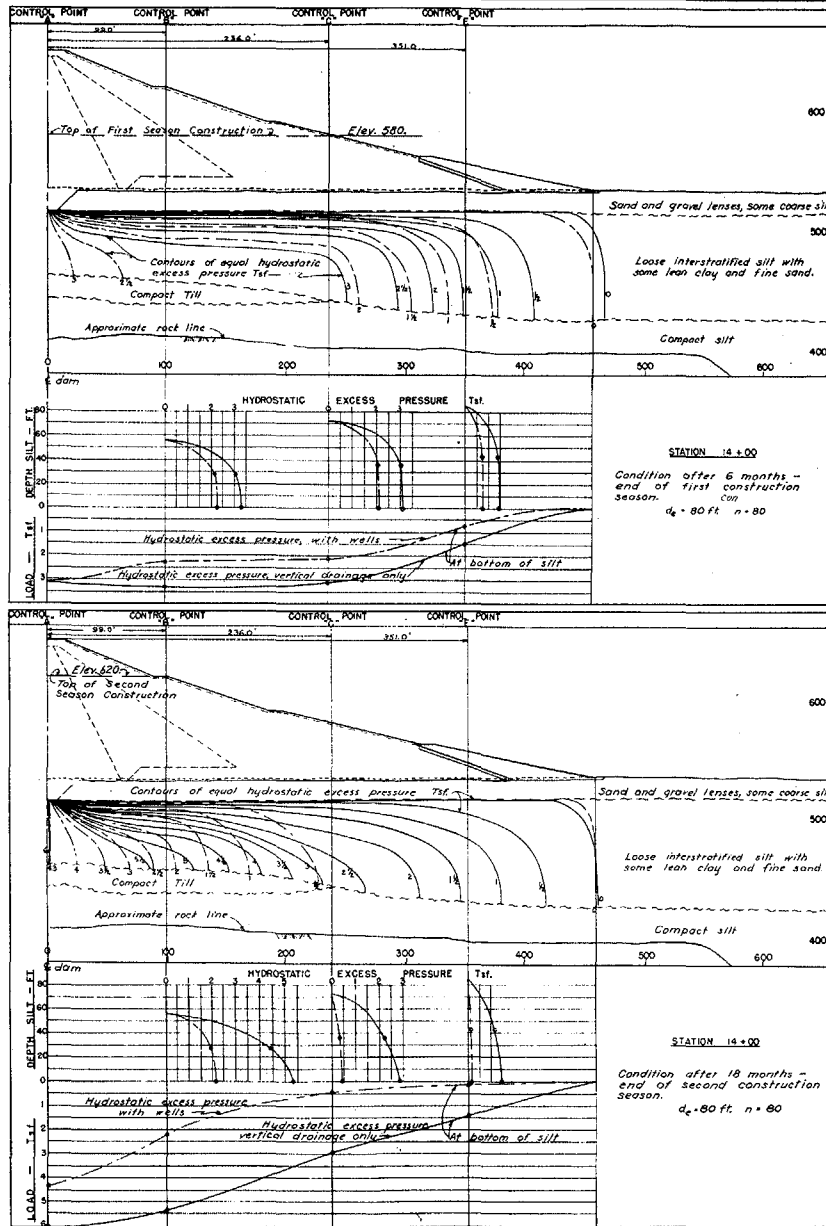


Studies made for 12 inch diameter diaphragm wells placed at vertices of 80 foot equilateral triangles.

Detailed computations given for one arc only, Circle No. 17.

CONNECTICUT RIVER FLOOD CONTROL	
CLAREMONT DAM	
STABILITY ANALYSIS - CIRCLE NO. 17	
SUGAR RIVER	NEW HAMPSHIRE
IN SHEETS	SCALE AS SHOWN
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945	
SUBMITTED	SOILS LABORATORY STUDY
HEAD, DIST. ENGINEER	PREPARED BY
DESIGNED BY	TRACED
CHECKED BY	FILE NO.

CL A. OF D.



NOTE:

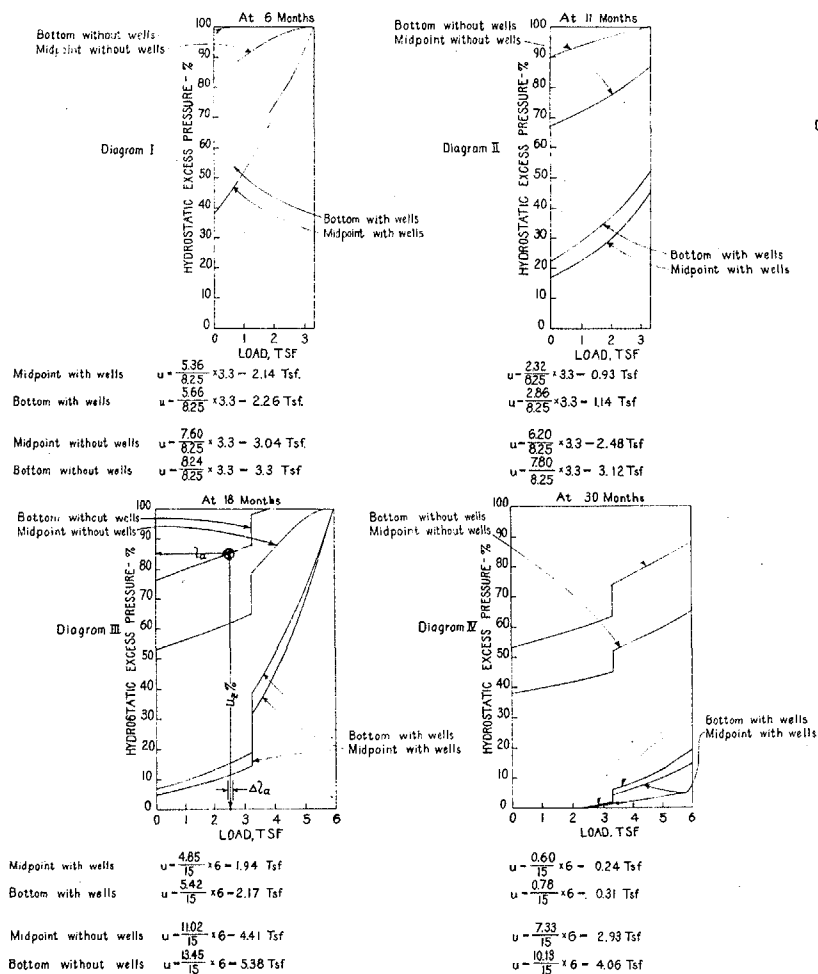
Hydrostatic excess pressure with wells - consolidation by radial flow to wells and by single vertical drainage
 Hydrostatic excess pressure without wells by single vertical drainage only. Base of consolidating soil considered resting on an impervious material

Drain wells - well diameter $d_w = 1$ ft. $n = \frac{d_c}{d_w} = 80$
 well spacing $d_c = 80$ ft.

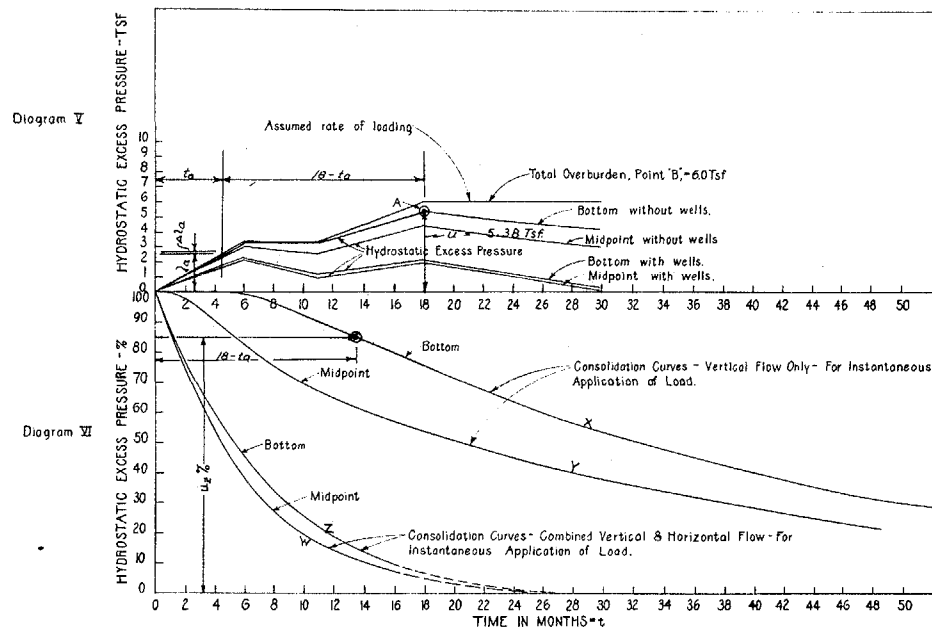
KEY	DATE	REVISION (Indicated by Δ)	REVIEW	CHK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL	
CLAREMONT DAM	
CONTOURS OF HYDROSTATIC EXCESS PRESSURE	
SUGAR RIVER	NEW HAMPSHIRE
IN 1 SHEETS	SHEET NO. 1
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN 1945	
SUBMITTED	SOILS LABORATORY STUDY
DESIGNED BY	TRACED
CHECKED BY	FILE NO.

PLATE NO. A25



u - Hydrostatic Excess Pressure, Per Unit Area.



Typical computation to determine point, as 'A', in Diagram V:

Point 'A' represents 'u', hydrostatic excess pressure per unit area, at bottom of soft silt, at 18 months, for condition of vertical drainage only. Computations will be made with help of Diagram III, marked 'At 18 Months'. Abscissas in Diagram III represent dam overburden which varies at the rate indicated in Diagram V by curve labeled 'Assumed rate of loading'.

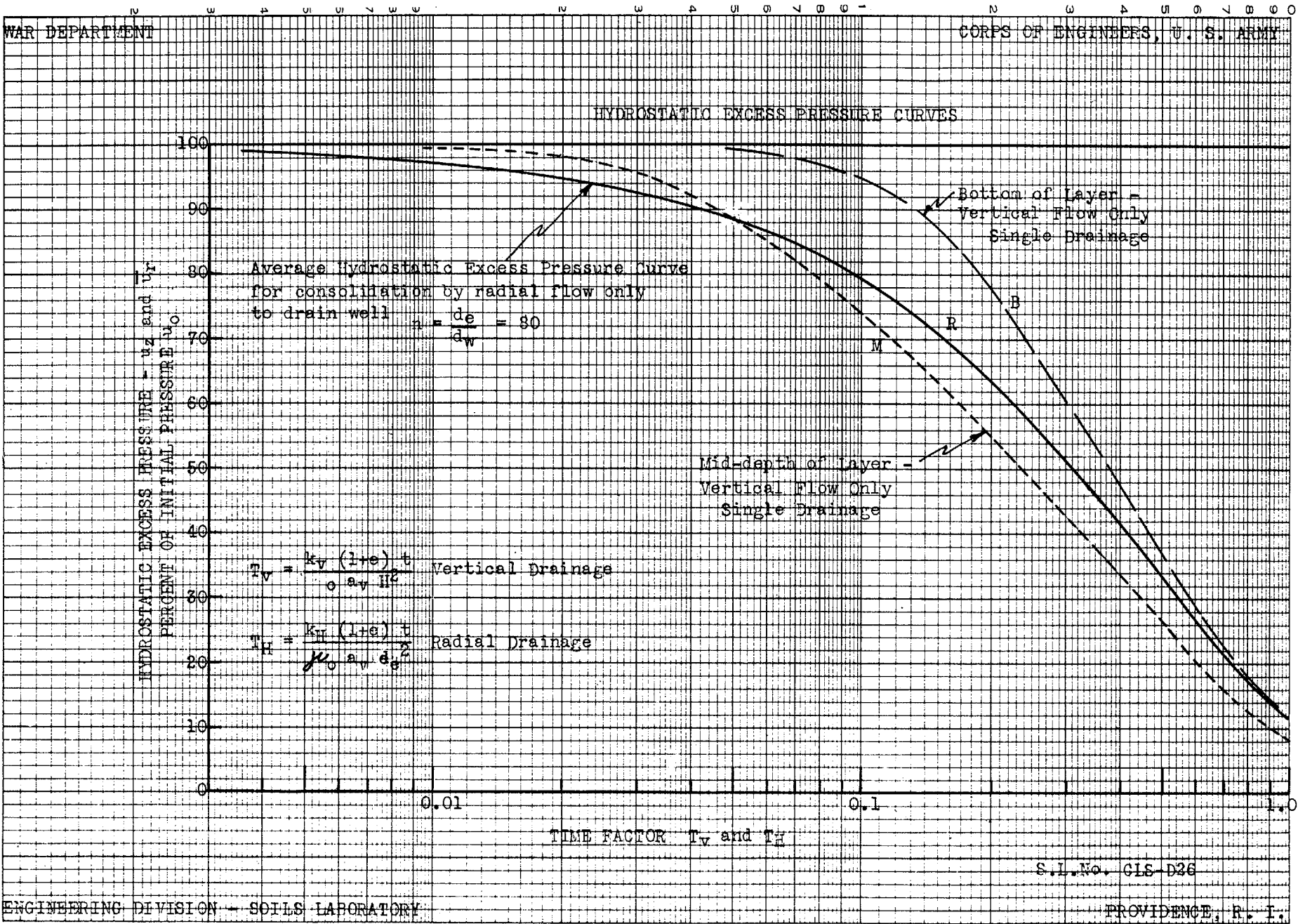
Let ΔI_a represent increment of load added at time in months = t_a .
Then ΔI_a will exist as a consolidation load for time in months = $18 - t_a$.
And in Diagram VII, u_2 % = ordinate to consolidation curve for vertical flow only, bottom, at time $18 - t_a$.
This value of u_2 % is then plotted as an ordinate in Diagram III against its corresponding ΣI_a value for time t_a .

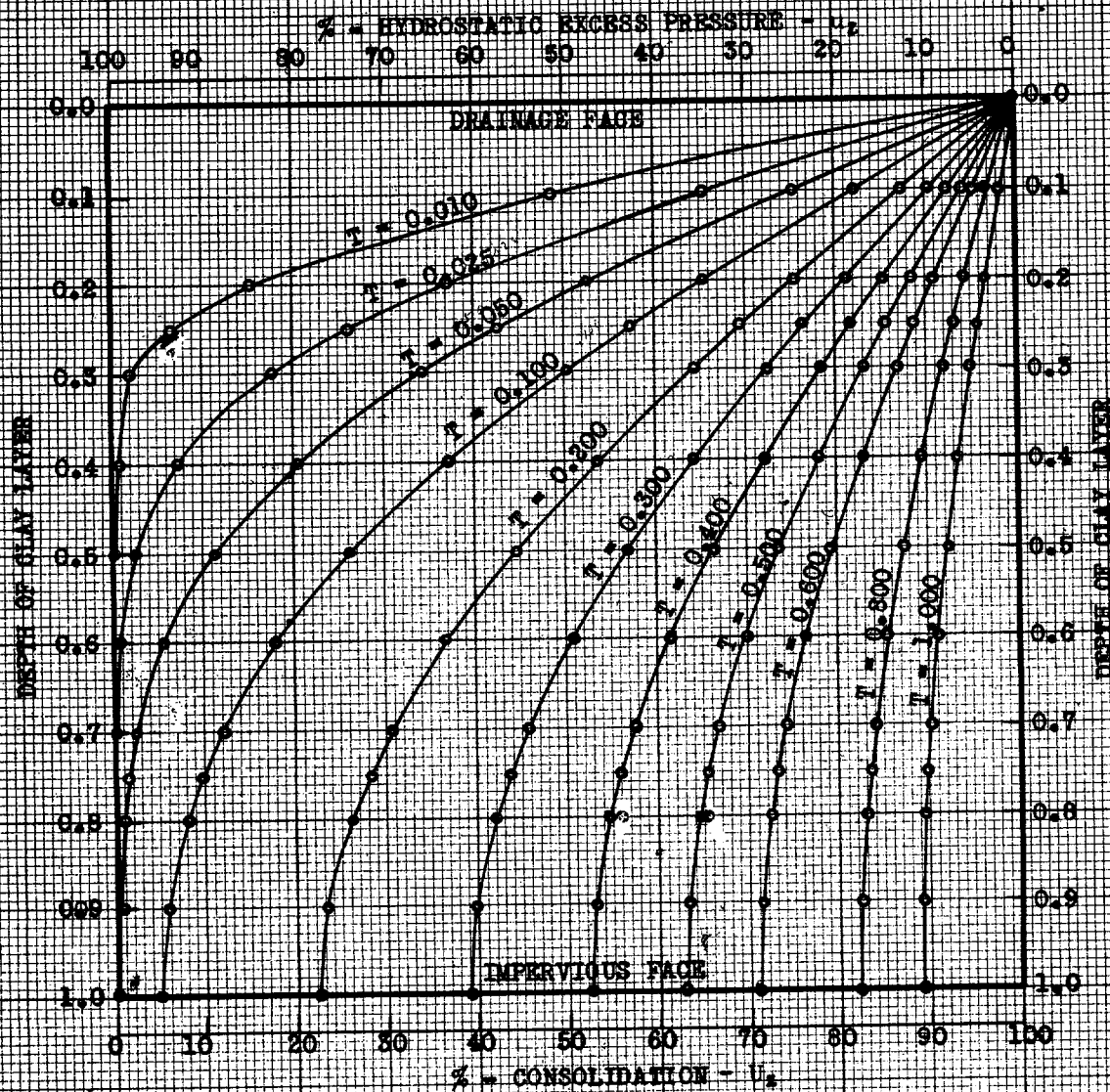
Total "u" for point "A" is then found by integrating area under curve, and multiplying ratio of that area to total area of Diagram III, by applied load at Point "A". This integration is done by a planimeter and result plotted in Diagram V as Point "A", $u = 5.38$ Tsf.

Effect of Drain Wells on Consolidation at Control Point "B"
99 Ft. Right Δ , Station 14+00. Drain Wells Spaced At Vertices
of 80 Ft. Equilateral Triangles. Depth of Soft Interstratified
Silt = 56 Ft. Computations Based on Values For Average Radial
Consolidation and Vertical Consolidation For Depth Considered.

CLAREMONT DAM.
GRAPHICAL DETERMINATION OF CONSOLIDATION
FOR NON - UNIFORM RATE OF LOADING

										IN SHEETS		SHEET NO.	
										U.S. ENGINEER OFFICE, PROVIDENCE, R.I.,		Scales AS SHOWN	
										U.S. ENGINEER OFFICE, PROVIDENCE, R.I.,		SOILS. LABORATORY STUDY	
										PROJECTED 1st Series		ENGINEER	
										HEAD SOILS LABORATORY		SL. NO. GLS - D7	
										REMARKS 1st Series		DRAWN: 22-2 TRACED: 22-2	
										KEY DATE		FILE NO.	
										REVISION (indicated by Δ)		REVIEW BY	
										AP BY			

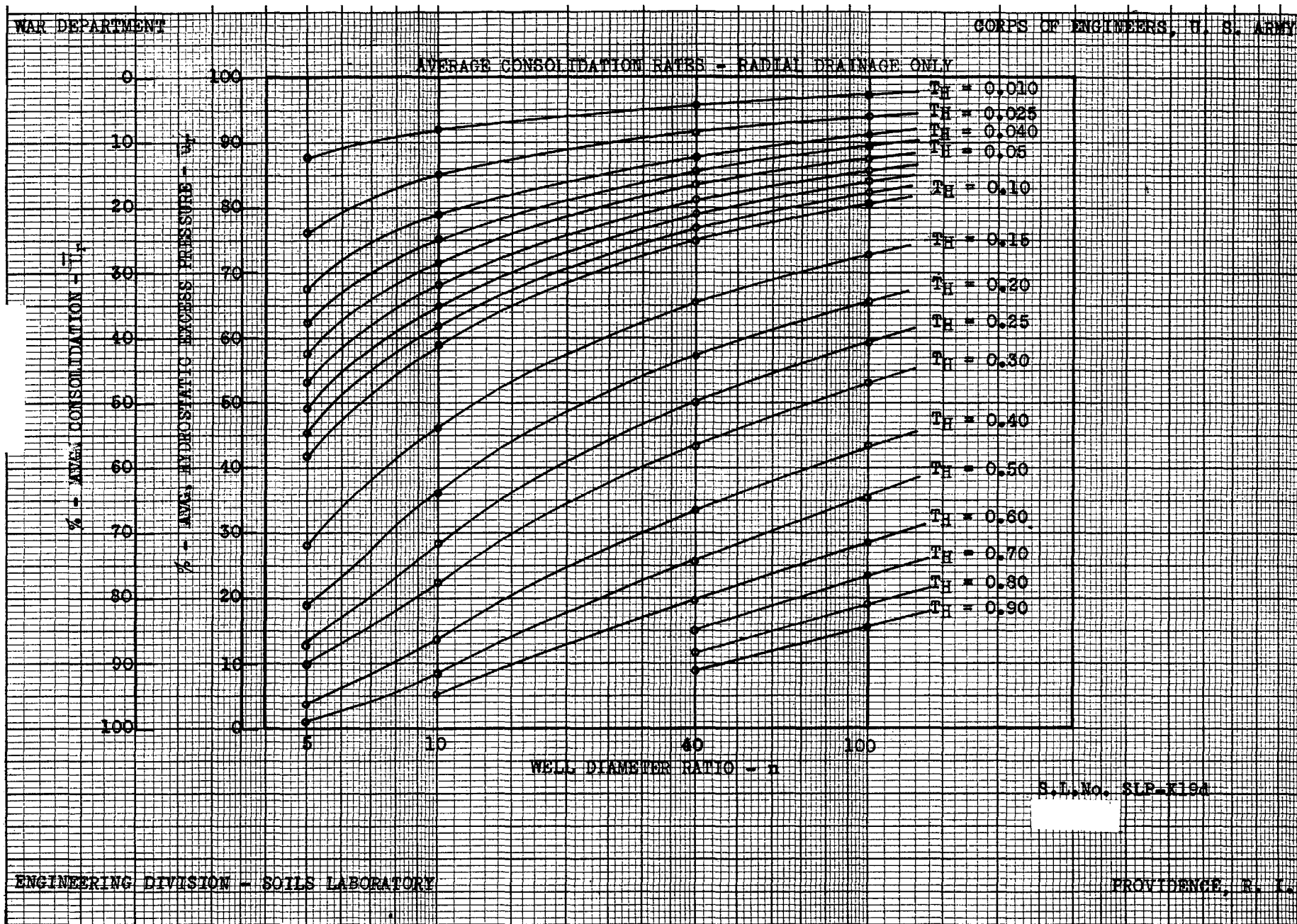


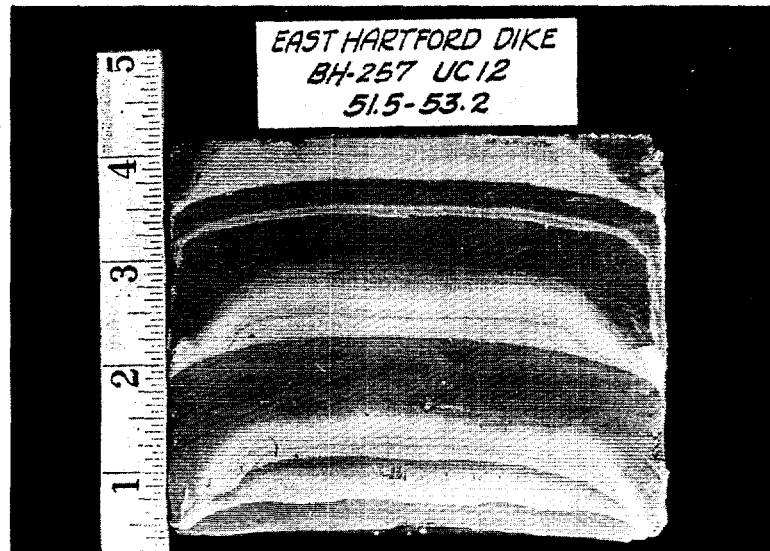


Variation of Consolidation for Given Time Factor at Given Depths. Consolidation by vertical flow only.

$$T = T_v$$

S.L.No. SLP-104





EAST HARTFORD DIKE

SLD 405

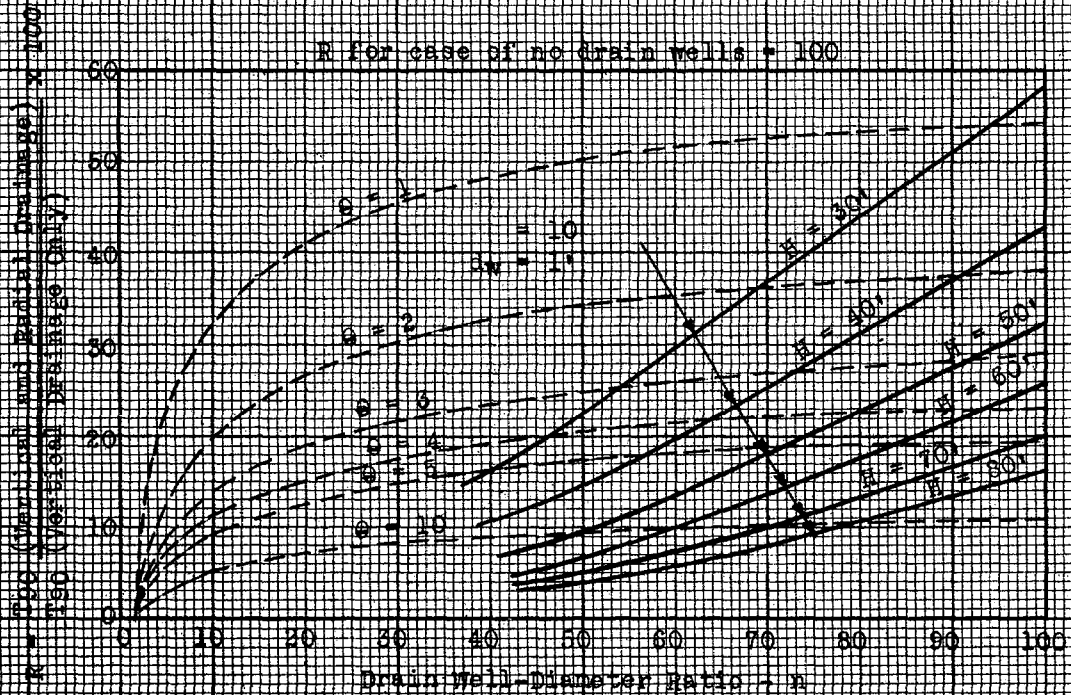
BH-257, UC12 - Depth 51.5 - 53.2

Alternate layers of fine silt and fatty clay -
slightly irregular in pattern. Distorted by early
type soil sampler.

SMEARED VARVED SOIL SAMPLE

CL. A. of D.

Plate No. A30



$$\frac{\text{Diameter of Influence Zone}}{\text{Drain Well Diameter}} = \frac{R_d}{R_w} = \alpha$$

$$\alpha = \frac{R_d}{R_w} = 1.0$$

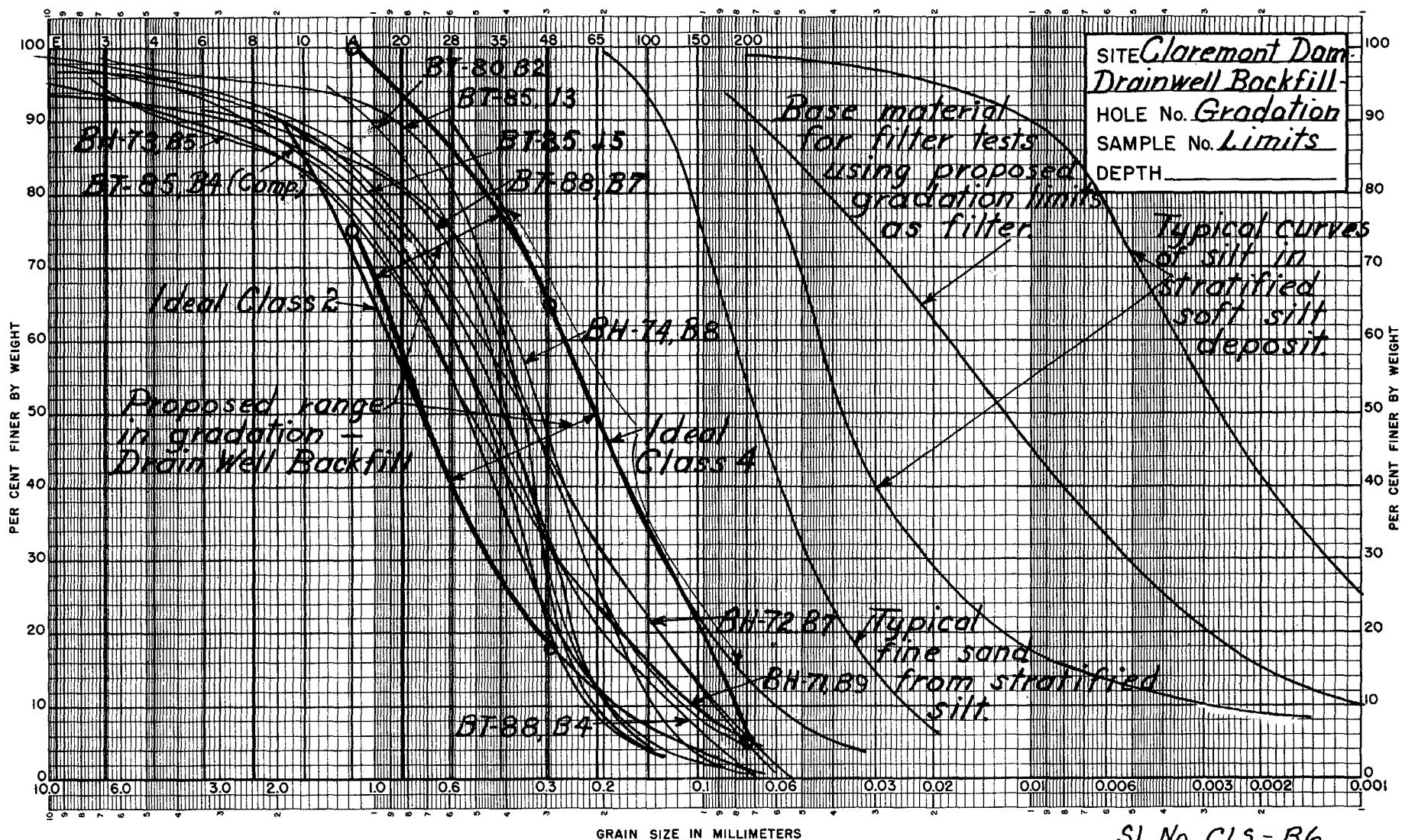
H = Thickness of Consolidating Layer - Single Drainage Surface

$$c = \frac{H^2 k_v}{C_e^2 k_v}$$

EFFECT OF COMBINED DRAINAGE ON T_{90}
 FOR INSTANTANEOUS LOADING

S.I. No. CLS-D25

PLATE NO. A32



SL.No. CLS - B6

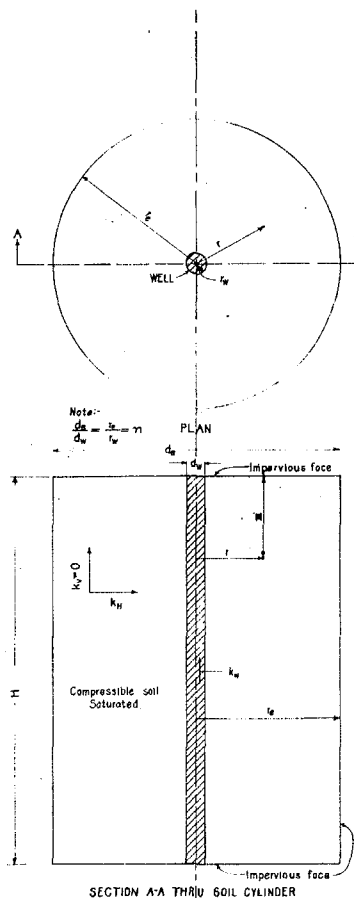
Gravel	Coarse Sand	Medium Sand	Fine Sand	Coarse Silt	Medium Silt	Fine Silt or Coarse Clay
Class 1	Class 2		Class 6		Class 10 or 10c	
Class 3		Class 4		Class 8		Class 12 or 12c
Class 5		Class 7		Class 9		Class 11
Class 13 or 13c		Class 13 or 13c		Class 13 or 13c		Class 13 or 13c

SOILS LABORATORY

MECHANICAL ANALYSIS

PROVIDENCE, R.I.

CL A. OF D.



ASSUMPTIONS

1. No flow in soil in vertical direction.
2. Radial flow in soil.
3. Outer surfaces of soil cylinder impervious.
4. Strain in soil due to consolidation occurs only in vertical direction and is uniform at any depth "z" over entire soil cylinder at any time "t".
5. Only vertical flow occurs in well - minor radial flow neglected.
6. Presence of well does not relieve soil of any load during process of consolidation.
7. Uniform surface loading - u_0

BOUNDARY CONDITIONS

1. No vertical flow in soil: $k_v = 0$
2. No radial flow at outer surface: $du/dr = 0$ at $r = r_w$
3. Excess water pressure in soil, u_{ws} , is equal to excess water pressure in well, u_{wz} , at $r = r_w$.
4. Radial flow from soil into well at any depth "z" is equal to change of seepage up the well:

$$2\pi r_w k_v \frac{du_{wz}}{dz} dz = -\pi r_w^2 h_w \frac{d^2 u_{wz}}{dz^2} dz.$$
5. Excess water pressure in well, u_{wz} , is zero at $z = 0$.
6. No vertical flow in well at depth "z": $du_{wz}/dz = 0$.
7. Because of assumption (4) the rate of void ratio change at any depth "z" is equal to rate of change of average excess water pressure in soil at any depth "z" multiplied by c_v , coefficient of compression of the soil:

$$\frac{du_{wz}}{dz} = \frac{\partial u_{ws}}{\partial t} c_v$$
8. At time "t" = 0, the average hydrostatic excess pressures at all depths equal the load - or $u_0 = u_{ws}$.

BASIC PARTIAL DIFFERENTIAL EQUATIONS AND SOLUTIONS

1. Because of assumption (4) the amount of water from soil between r_w and r flowing by surface having radius r is equal to total change of void ratio of soil between r_w and r :

$$\frac{h_w}{c_v} \frac{\partial u_{ws}}{\partial t} 2\pi r dr = -\frac{\partial}{\partial r} (q^2 - r^2) \pi \frac{du_{ws}}{dr} = -\frac{\partial}{\partial r} (q^2 - r^2) \pi \frac{\partial u_{ws}}{\partial t}$$
2. Hydrostatic excess pressure at any point, u_{ws} , is

$$u_{ws} = \frac{c_v}{1 - e} \frac{\partial u_{ws}}{\partial t} + \frac{1}{2(1 - e)} \frac{\partial}{\partial t} \left(\frac{r^2}{h_w} - \frac{r_w^2}{h_w} \right) + u_{wz}$$
3. Average hydrostatic excess pressure in soil at any depth "z" is

$$u_0 = \frac{c_v}{1 - e} \frac{\partial u_0}{\partial t} + \frac{1}{2(1 - e)} \frac{\partial}{\partial t} \left[\frac{r^2}{h_w} \ln \left(\frac{r}{r_w} \right) - \frac{3r^2 - 1}{4r^2} \right] + u_{wz}$$

Set function in brackets equal to $F(r)$
4. Meeting boundary conditions number 4, 5, 6 and 8 and making use of equations 1, 2, and 3 the hydrostatic excess pressure in well at time $t = 0$ is

$$u_{wz} = u_0 \left[1 - \frac{r^2 - r_w^2}{1 - e} \frac{\partial}{\partial t} \right] \text{ where } \beta = \sqrt{\frac{2k_v (1 - e)}{h_w c_v F(r)}}$$
5. Solution for average hydrostatic excess pressure at any depth "z" and time "t" is

$$u_0 = u_0 e^{-\beta^2 F(r) t} \text{ where } F(r) = \frac{r^2 - r_w^2}{1 - e} \frac{\partial}{\partial t} \text{ and } \beta = \sqrt{\frac{2k_v (1 - e)}{h_w c_v F(r)}}$$
6. Solution for hydrostatic excess pressure at any point, u_{ws} , and time "t" is

$$u_{ws} = u_0 \left[\frac{F(r)}{F(r)} \left(\ln \frac{r}{r_w} - \frac{r^2 - r_w^2}{2r^2} \right) + 1 - F(r) \right]$$
7. Solution for hydrostatic excess pressure in well at any depth "z" and time "t" is

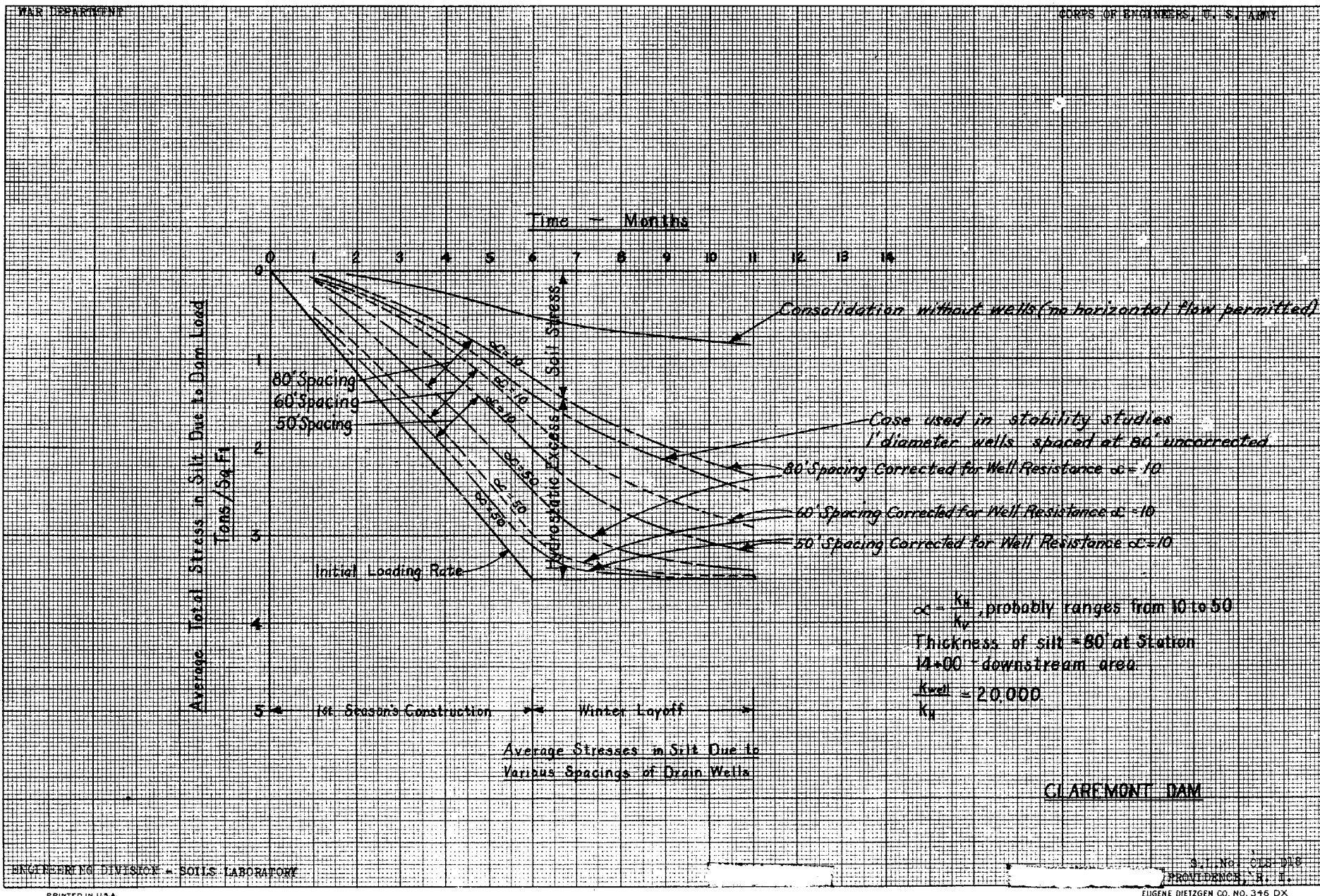
$$u_{wz} = u_0 [1 - F(r)]$$
8. Overall average hydrostatic excess pressure in soil cylinder is

$$\bar{u} = \frac{1}{\pi r_w^2} \int_0^{r_w} u_{ws} 2\pi r dr = \text{solve by approximate method - using Simpson's rule.}$$

PLATE NO. A-33

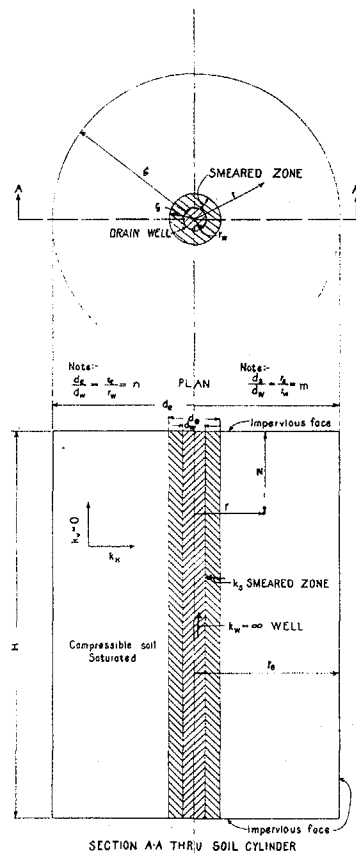
DRAIN WELL THEORY FORMULAE FOR CONSOLIDATION BY RADIAL DRAINAGE TO DRAIN WELL - UNIFORM VERTICAL STRAIN - WELL RESISTANCE CONSIDERED	
IN 1 SHEETS	SHEET NO. 1
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., DEC. 1944	
SUBMITTED BY <i>J. S. Lane</i>	ENGINEER
HEAD SOILS LABORATORY	SOILS LABORATORY STUDY
PREPARED BY <i>J. S. Lane</i>	DESIGNED BY <i>J. S. Lane</i>
TRACED BY <i>J. S. Lane</i>	CHECKED BY <i>J. S. Lane</i>
SL NO. CLS-DIO	FILE NO.

KEY	DATE	REVISION	BY	APPROVED



CL A. OF D.

PLATE NO. A-35



ASSUMPTIONS

1. No flow in soil in vertical direction.
2. Radial flow in soil.
3. Strains in soil due to consolidation occur only in vertical direction.
4. Difference in strains which tend to develop shear strain have no effect upon rate of consolidation.
5. Well permeability very high $k_w \rightarrow \infty$
6. Smear occurs around well before load added and that after smearing no change in void ratios occurs in smeared zone. Permeability in smeared zone reduces to k_s .
7. Initial hydrostatic excess pressure in water u_0 from load uniform thru-out soil cylinder at time $t=0$
8. Presence of well does not relieve soil of any load during process of consolidation.

BOUNDARY CONDITIONS

1. Outer surface is impervious $\therefore \frac{\partial u}{\partial r} = 0$ at $r = r_0$.
2. No vertical flow $\therefore \frac{\partial u}{\partial z} = 0$ at all points.
3. Hydrostatic excess pressure, u , in undisturbed soil between r_0 and r_s , and hydrostatic excess pressure, u_s , in smeared zone between r_s and r_w are equal at $r = r_s$.
4. Flow from undisturbed soil across boundary at $r = r_s$ is equal to flow into smeared zone

$$k_w \frac{\partial u}{\partial r} 2\pi r_s = k_s \frac{\partial u}{\partial r} 2\pi r_s$$

5. To meet assumption number 6 the excess pressure at any point "r" in smeared zone is

$$u_s = u_0 \frac{\ln(r/r_w)}{\ln(r_s/r_w)}$$

BASIC PARTIAL DIFFERENTIAL EQUATIONS AND SOLUTIONS

1. Rate of flow out of a small volume of soil is equal to rate of volume change.

$$\frac{k_s}{r} \left[\frac{1}{r} \frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2} \right] 2\pi r dr dz = - \frac{\partial u}{\partial t} \frac{2\pi r dr dz}{1+e} = - \frac{\partial u}{\partial t} \frac{2\pi r dr dz}{1+e}$$

2. Solution of above equations for average hydrostatic excess pressure subject to boundary conditions is

$$u_s = \frac{4u_0}{\pi} \sum_{n=1}^{\infty} \frac{J_0(\alpha_n r/r_w)}{\alpha_n J_1(\alpha_n r/r_w)} e^{-\alpha_n^2 T_v} \quad d) T_v = \frac{4k_s (1+e) t}{\mu \alpha_n^2 r_w^2}$$

- a) Where α is a root of

$$\frac{k_s}{\alpha k_w m \ln(r_s/r_w)} \left[J_0(\alpha r/r_w) + U_1(\alpha r/r_w) \right] = 0$$

- b) $U_0(\alpha r/r_w) = J_0(\alpha r/r_w) Y_0(\alpha r/r_w) - J_0(\alpha r/r_w) Y_0(\alpha r/r_w)$
- c) $U_1(\alpha r/r_w) = J_1(\alpha r/r_w) Y_0(\alpha r/r_w) - J_0(\alpha r/r_w) Y_1(\alpha r/r_w)$ Bessel equations

3. Hydrostatic excess pressure at any point $r > r_s$

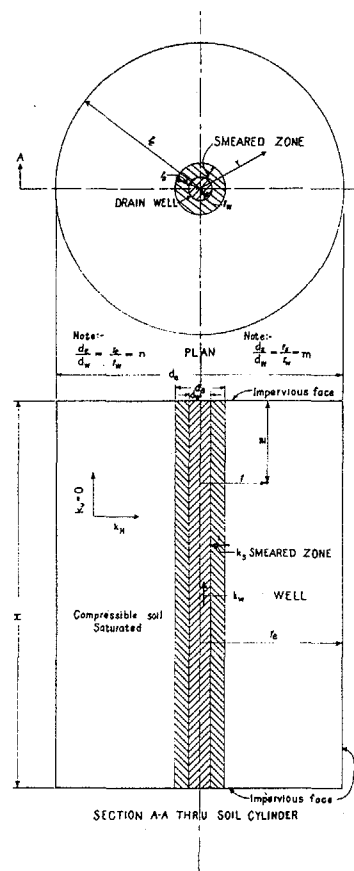
$$u_s = -2u_0 \sum_{n=1}^{\infty} \frac{J_0(\alpha_n r/r_w) [J_0(\alpha_n r/r_w) Y_0(\alpha_n r/r_w) - J_0(\alpha_n r/r_w) Y_0(\alpha_n r/r_w)]}{\alpha_n^2 J_1^2(\alpha_n r/r_w)} e^{-\alpha_n^2 T_v}$$

DRAIN WELL THEORY FORMULAE FOR CONSOLIDATION BY RADIAL DRAINAGE TO DRAIN WELL- SMEAR EFFECT CONSIDERED

IN 1 SHEETS		SHEET NO. 1	
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., DEC. 1944			
SUBMITTED <i>H. L. ...</i>		ENGINEER <i>H. L. ...</i>	
HEAD, SOILS LABORATORY		SOILS LABORATORY STUDY	
PREPARED BY <i>H. L. ...</i>	DESIGNED BY <i>H. L. ...</i>	FILE NO.	S. L. NO. CLS - 011

KEY	DATE	REVISION (Indicated by Δ)	REVIEWED BY

CL A. OF D.



ASSUMPTIONS

1. No flow in soil in vertical direction.
2. Radial flow in soil.
3. Outer surface of soil cylinder impervious.
4. Strains in soil due to consolidation occur only in vertical direction, and are uniform at any depth "z" over entire soil cylinder at any time "t".
5. Only vertical flow occurs in well - minor radial flow neglected.
6. Smear occurs around well before load added, and after formation of smear no change in void ratio occurs in smeared zone.
7. Presence of well does not relieve soil of any load during process of consolidation.

BOUNDARY CONDITIONS

1. Outer cylindrical surface is impervious, $\frac{\partial u_a}{\partial r} = 0$ at $r = r_0$.
2. No vertical flow: $v_z = 0$.
3. Average hydrostatic excess pressure, u_a , in soil is equal to uniform loading, u_0 , at time "t" = 0.
4. Hydrostatic excess pressure, u_a , in undisturbed soil between r_0 and r_s , and hydrostatic excess pressure, u_a , in smeared zone between r_s and r_w are equal at $r = r_s$.
5. Flow from undisturbed soil across boundary at $r = r_s$ is

$$k_0 \frac{\partial u_a}{\partial r} 2\pi r_s = k_s \frac{\partial u_a}{\partial r} 2\pi r_s$$
6. To meet assumption number 6, the hydrostatic excess pressure in smeared zone between r_s and r_w is

$$u_a = (u_{a0} - u_{a0}) \frac{\ln(r/r_s)}{\ln(r_w/r_s)}$$
7. Radial flow from any soil into well at any depth "z" is equal to change of seepage up the well

$$2\pi r_w k_s \frac{\partial u_a}{\partial z} dz = -\pi r_w^2 k_0 \frac{\partial^2 u_a}{\partial z^2} dz$$
8. Hydrostatic excess pressure in well, u_{a0} , is zero at "z" = 0.
9. No vertical flow occurs in well at depth "z" = H: $\partial u_{a0} / \partial z = 0$ at $z = H$.
10. Because of assumption number 4 the rate of void ratio change at any depth "z" is equal to rate of change of average hydrostatic excess pressure in soil at that depth, multiplied by α , coefficient of compressibility of the soil, $\partial e / \partial t = \alpha \partial u_a / \partial t$.

BASIC PARTIAL DIFFERENTIAL EQUATIONS AND SOLUTIONS

1. Because of assumption (4) the amount of water from soil between r_0 and r flowing by surface having radius r is equal to total change of void ratio of soil between r_0 and r .

$$\frac{k_0}{r} \frac{\partial u_a}{\partial r} 2\pi r dr = -\frac{\partial}{\partial t} (e - e_0) \pi r^2 \frac{\partial u_a}{\partial z} dz = -\frac{\partial}{\partial t} (e - e_0) \pi r^2 \frac{\partial u_a}{\partial z} dz$$
2. Hydrostatic excess pressure at any point r between r_0 and r_s is

$$u_a = \left(\frac{u_{a0}}{r_0} \frac{r}{r_0} \right) \frac{1}{2\pi (r_0^2 - r_s^2)} \left[\frac{r_0^2}{2} \ln \frac{r}{r_0} - \frac{r_s^2}{2} \ln \frac{r}{r_s} + \frac{r_s^2}{2} (r_0^2 - r_s^2) \ln(r_0) \right] + u_{a0}$$
3. Average hydrostatic excess pressure at any depth "z", between r_0 and r_s is

$$u_a = \left(\frac{u_{a0}}{r_0} \frac{r}{r_0} \right) \frac{1}{2\pi (r_0^2 - r_s^2)} \left[\frac{r_0^2}{2} \ln \frac{r}{r_0} - \frac{r_s^2}{2} \ln \frac{r}{r_s} + \frac{r_s^2}{2} (r_0^2 - r_s^2) \ln(r_0) \right] + u_{a0}$$
 — Set item in brackets equal to $F(r)$
4. Meeting boundary condition number 7, and making use of equations 1, 2, and 3 the hydrostatic excess pressure in well at time "t" = 0 is

$$u_{a0} = u_0 \left[1 - \frac{e^{-\alpha u_{a0}}}{1 + e^{-\alpha u_{a0}}} \right] \text{ where } \alpha = \frac{2k_0}{r_w^2} \frac{r_0^2 - r_s^2}{F(r_0)}$$
5. Solution for average hydrostatic excess pressure at any depth "z" and time "t" is

$$u_a = u_{a0} \frac{F(r)}{F(r_0)} \text{ where } F(r) = \frac{r_0^2 - r_s^2}{2} \ln \frac{r}{r_0} - \frac{r_s^2}{2} \ln \frac{r}{r_s} + \frac{r_s^2}{2} (r_0^2 - r_s^2) \ln(r_0)$$
6. Solution for hydrostatic excess pressure at any point "z" between r_0 and r_s at time "t" is

$$u_a = \frac{F(r)}{F(r_0)} \left[\ln \frac{r}{r_0} - \frac{r_s^2}{r_0^2} \ln \frac{r}{r_s} + \frac{r_s^2}{r_0^2} \ln(r_0) \right] + 1 - F(r_0)$$
7. Solution for hydrostatic excess pressure in well at any depth "z" at time "t" is

$$u_{a0} = u_0 \left[1 - F(r_0) \right]$$
8. Overall average hydrostatic excess pressure in soil cylinder is

$$\bar{u} = \frac{1}{H} \int_0^H u_a dz = \text{Solve by approximate method using Simpson's Rule.}$$
9. For case of smear and no well resistance the above equations hold with $F(r) = F(r)$.
10. For case of no smear and well resistance the above equations hold with $F(r) = F(r)$. (See Plate No. A-36)

33

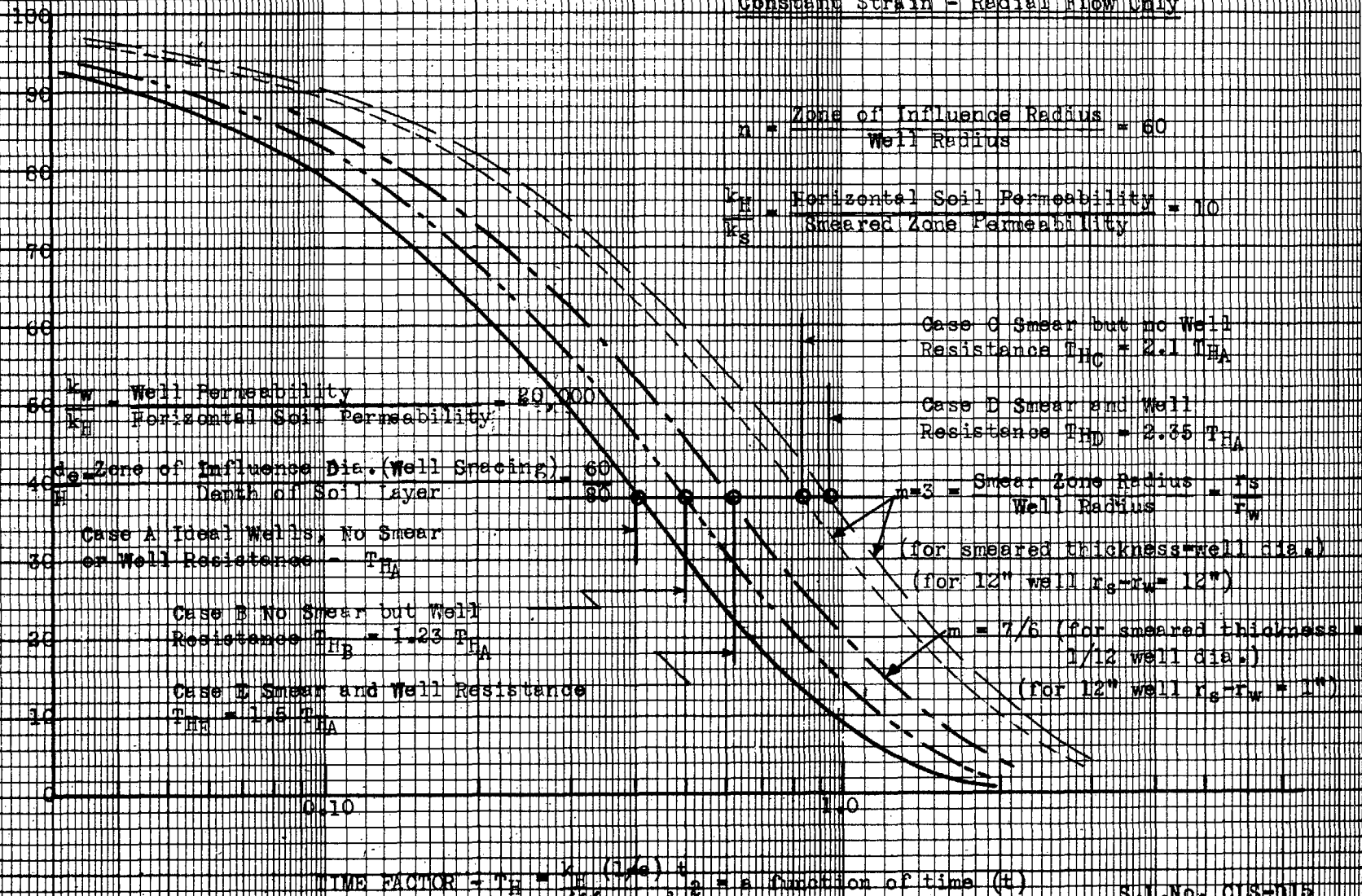
PLATE NO. A-36

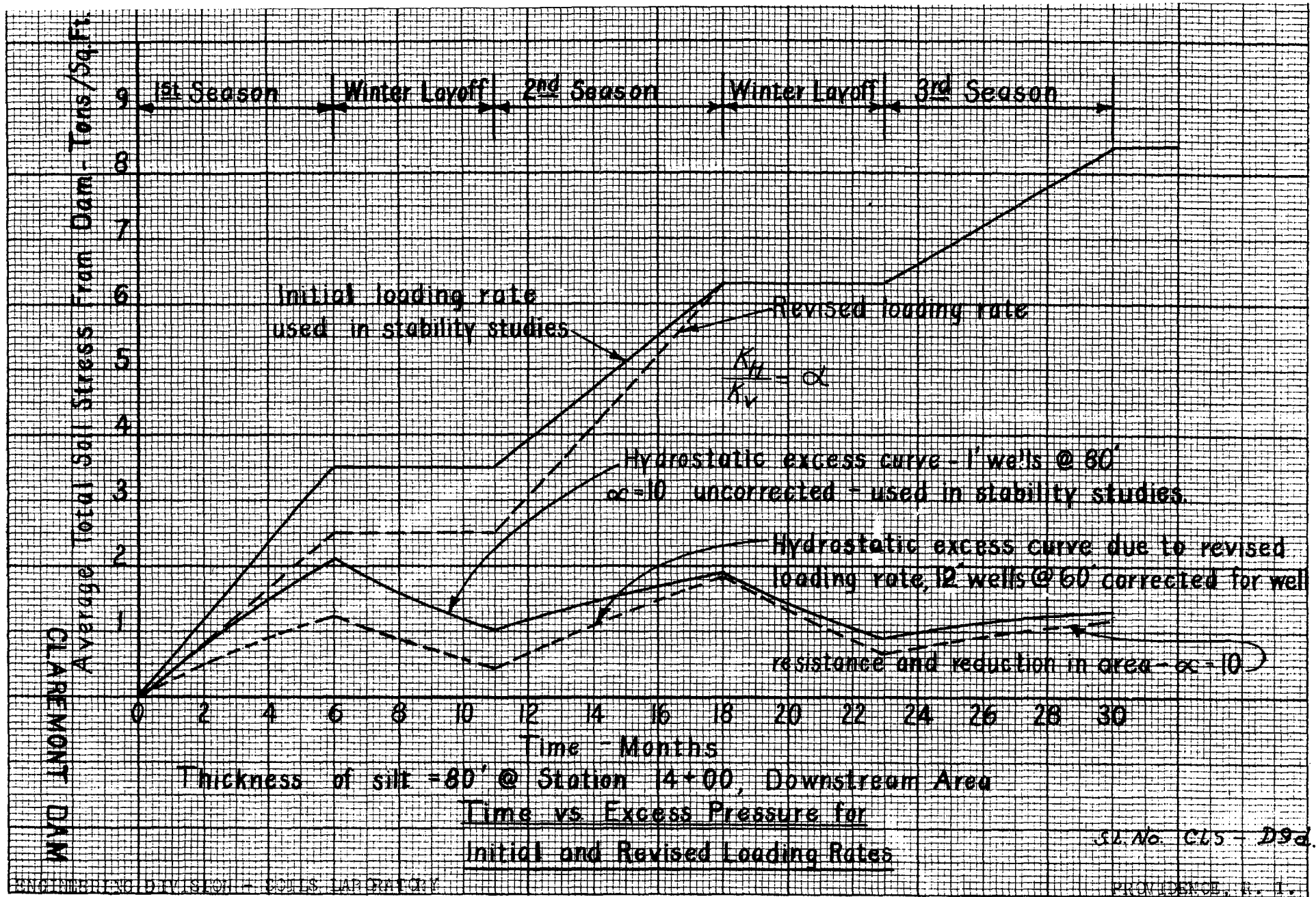
DRAIN WELL THEORY FORMULAE FOR CONSOLIDATION BY RADIAL DRAINAGE TO DRAIN WELL - UNIFORM STRAIN - SMEAR AND WELL RESISTANCE CONSIDERED	
IN 1 SHEETS	SHEET NO. 1
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN 1945	
SUBMITTED BY: <i>N. S. Lane</i>	
HEAD, SOILS LABORATORY	
DESIGNED BY: <i>N. S. Lane</i>	SOILS LABORATORY STUDY
CHECKED BY: <i>N. S. Lane</i>	TRACED
FILE NO.	FILE NO.

DATE: MAY 1945	REVISION (Indicated by Δ)	REWORK BY: AP BY
----------------	---------------------------	------------------

PLATE NO. A37

0 = Ratio of vertical average displacement of soil layer to initial thickness of soil layer







TYPICAL EXAMPLE
EXCESS PRESSURE IN PERVIOUS STRATA AT LANDSIDE TOE OF DIKE.

$$C = \frac{1}{\sqrt{1.4}} \cdot \sqrt{\frac{0.5 \times 10^{-3}}{50.0 \times 10^{-3}}} \times \frac{1}{20.725} = 3.54 \times 10^{-3}$$

$$CL_1 = 3.54 \times 10^{-3} \times 200 = 0.708$$

$$C = 0.1 \times 10^{-3} \times \frac{1}{200} = 0.591$$

$$C(2L_2 - L) = 3.54 \times 10^{-3} \times \frac{1000}{1} = 0.637$$

$$C(2L_2 - X) = C(2L_2 - L_1 - W) = 3.54 \times 10^{-3} \times \frac{1}{1720} = 0.610$$

$$e^{CL_1} = e^{0.708} = 2.03$$

$$e^{CX} = e^{0.591} = 2.60$$

$$C(2L_2 - W) = 0.637 = 1.89$$

$$e^{C(2L_2 - W)} = 0.610 = 1.84$$

$$\tanh_1(CL_1) = \tanh_1(0.708) = 0.6004$$

ASSUMPTIONS

1. Flow in pervious layer is horizontal.
2. Flow in flood plain silt blanket is :
 - a) Vertically down $0 < x < L_1$
 - b) Vertically up $L_1 < x < L_2$
3. No flow horizontally in pervious layer at L_2 .
4. Tailwater is at landside ground surface.

EXCESS HEADS IN PERVIOUS LAYER

$$h = (2a) \sinh^{-1} \left(\frac{x}{c} \right) + H. \text{----- Equation No.3}$$

$$h = a_1 \left[\frac{e^{cx}}{e^{(2cL_2 - cx)}} \right]^{L_1 \langle x \rangle L_2} \text{----- Equation No. 4.}$$

CONSTANTS FOR ABOVE EQUATIONS

$$a = \frac{H}{[1 - \tanh(C_L)] e^{C_L} + [1 + \tanh(C_L)] e^{(2C_L - C_L)}} 2 \cosh(C_L)$$

BASIC PARTIAL DIFFERENTIAL EQUATIONS

At any point $0 < x < L$,
 Rate of seepage increase in pervious layer = Inflow from
 riverside top blanket
 $-k_2 \frac{\partial^2 h}{\partial x^2} \times A_2 dx = +k_1 \frac{(H-h)}{A_1} dx \dots \text{Equation No.1}$

At any point $x, x < x_2$
Rate of seepage decrease in pervious layer. Outflow through
landside top layer.
 $k_2 \frac{\partial^2 h}{\partial x^2} A_1 dx = + k_1 h dx$ ----- Equation No.2.

Boundary Conditions for Partial Differential Equations

At

$x = 0, \quad h = H$

$x = L_1, \quad h(\text{by equation No.1}) = h(\text{by equation No.2})$

$x = L_1, \quad \frac{\partial h}{\partial x} (\dots) = \frac{\partial h}{\partial x} (\dots)$

$x = L_2, \quad \frac{\partial h}{\partial x} = 0$

Solution of above two partial differential equations give equations Nos. 3 and 4, subject to above boundary conditions.

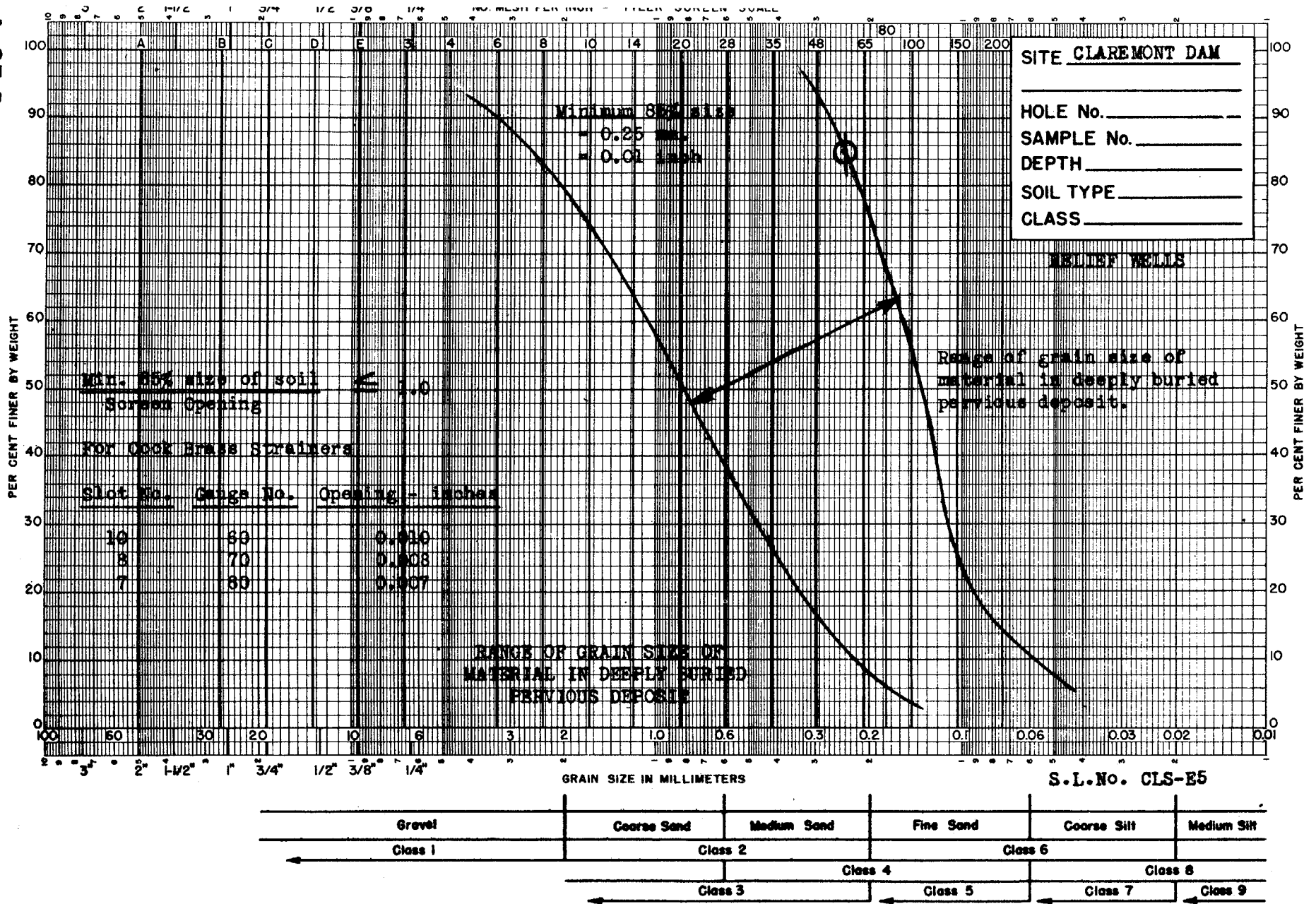
SEEPAGE EQUATION

$$q = \frac{k_g}{c A_1} (e^{(2cL_2 - cL_1)} - e^{cL_1}) = \text{Seepage through landside blanket per unit length of dike, per unit of time.}$$

FORMULA FOR SEEPAGE UNDER DWKE WITH NATURAL BLANKET OVER PERVIOUS LAYER	
IN 1 SHEET	SHEET NO. 1
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., DEC. 1944	
SUBMITTED <i>N. G. Jones</i> (ENGINEER) HEAD, SOIL LABORATORY	SOILS LABORATORY STUDY
PREPARED BY <i>W. A. Brown</i>	GRAPHIC: B.D.L. TRACED CHECKED: B.A.R. FILE NO.
	3L NO. W-8-AD

A. OF D.

PLATE NO. A40

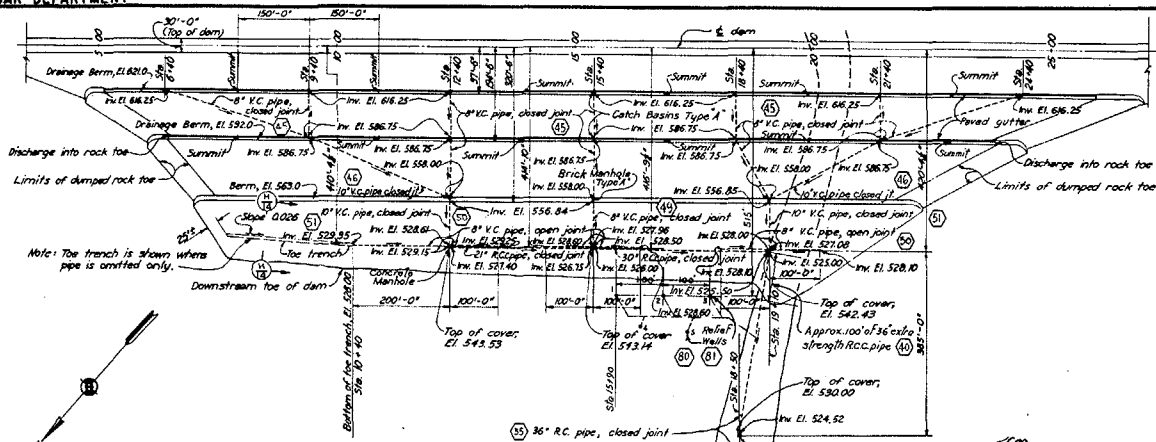


SOILS LABORATORY, ENGINEERING DIVISION

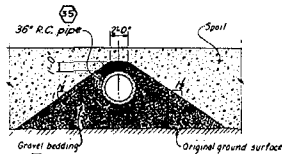
MECHANICAL ANALYSIS

PROVIDENCE, R. I.

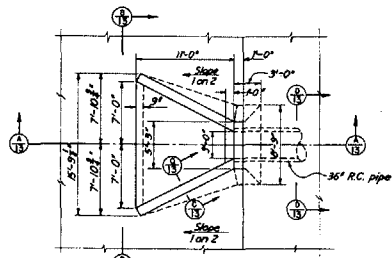
CL. A. OF D.

PLAN OF DOWNSTREAM SLOPE
SHOWING DRAINAGE STRUCTURES

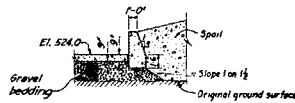
SCALE: 1" = 100'



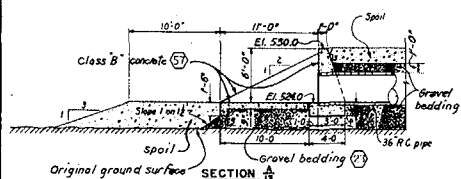
SECTION A-A



PLAN



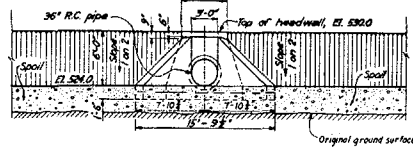
SECTION B-B



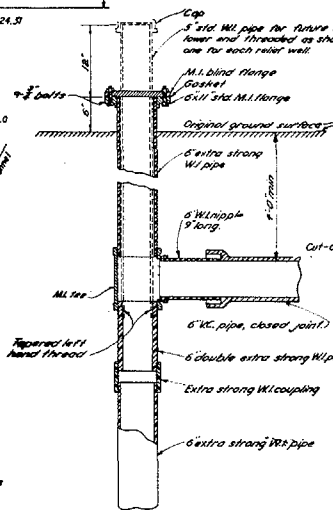
SECTION C-C

HEADWALL DETAILS

SCALE: 1/2\"/>

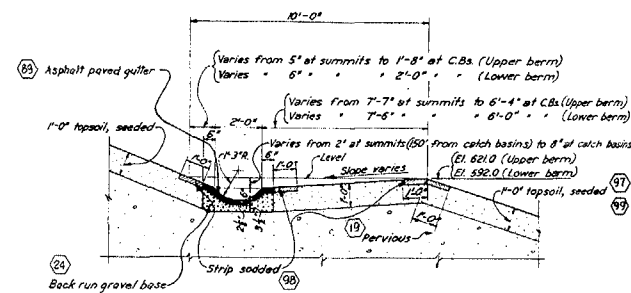


SECTION D-D



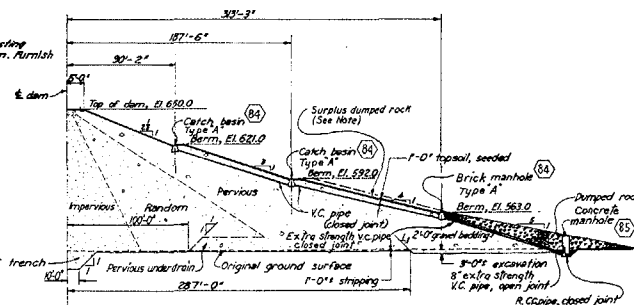
RELIEF WELL DETAIL

SCALE: 1/2\"/>



DRAINAGE BERM AND GUTTER DETAILS

SCALE: 1/2\"/>



SECTION THRU DOWNSTREAM SLOPE DRAINAGE STRUCTURES

SCALE: 1/2\"/>

NOTES:

- Elevations refer to Mean Sea Level Datum.
For catch basin and manhole details see Sheet No. 14.
If available, surplus rock from excavations and borrow pits shall be dumped on the downstream surface of the dam between El. 526.0 and El. 522.0.
Figures in hexagons indicate item numbers under which payment will be made.
Elevations indicated on the drawings are the required ultimate elevations. Necessary adjustments for estimated settlement will be made by the contracting officer prior to construction.
All pipe under rock toe shall be extra strength pipe.
21-inch R.C.C. pipe and 30-inch R.C.C. pipe will be paid for under items 38 and 39 respectively.
The location of relief wells 4 and 5 is subject to change.
For location of drain wells see Sheet No. 10.

NOTE:
Drain wells will be used under dam in area covered by first seasons fill where soft interstratified silt is 15'-0\"/>

DRAIN WELL DETAILS

NOT TO SCALE

KEY	DATE	REVISION (Indicated by Δ)	REVIEWED BY	APPROVED BY

CONNECTICUT RIVER FLOOD CONTROL
CLAREMONT DAM
EMBANKMENT DRAINAGE SYSTEM
PLAN AND DETAILS

SUGAR RIVER NEW HAMPSHIRE

SCALE: 1" = 100 FT.

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED	APPROVED	RECOMMENDED	APPROVED

PLATE NO. A41